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1980

Petrolia Dam,
Winnett, Montana,
Petroleum County,
MT 00008

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PHASE I INSPECTION REPORT
NATIONAL DAM SAFETY PROGRAM

PETROLIA DAM
WINNETT, MONTANA
PETROLEUM COUNTY
MT 00008

prepared for

HONORABLE THOMAS L. JUDGE
GOVERNOR, STATE OF MONTANA

MONTANA DEPARTMENT OF NATURAL
RESOURCES AND CONSERVATION
(OWNER - ADMINISTRATOR)

PETROLIA WATER USERS' ASSOCIATION
(OPERATOR)

prepared by

HKM ASSOCIATES
BILLINGS, MONTANA

DECEMBER 1980

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- Petrolia Water Users' Association Comments on the Draft Report.
- Montana DNRC Inspection Report, December 5, 1980.

EXECUTIVE SUMMARY

Personnel of HKM Associates, under a contract with the Montana Department of Natural Resources and Conservation (DNRC), and with representation from the Montana DNRC and the Petrolia Water Users' Association, inspected Petrolia Dam on September 26, 1979. The inspection and evaluation was performed under the authority of Public Law 92-367. Petrolia Dam is located on Flatwillow Creek, approximately 7 miles southeast of Winnett, Petroleum County, Montana. A contract for dam construction was awarded in June 1950, and the work was completed in August 1951.

FINDINGS AND EVALUATION

Petrolia Reservoir stores runoff from a total drainage area of 612 square miles. There is one other major storage project in the watershed, and that is Yellow Water Reservoir (storage to first overtopping elevation of 7700 acre-feet (AF)). The Yellow Water Storage Project is located approximately 13 miles upstream of Petrolia Reservoir, and has a contributory watershed of 55 square miles.

The Petrolia Storage Project is multi-purpose in nature as the stored water is used for irrigation and occasionally for recreation. Also, storage is provided for sediment accumulation and floodwater detention. Active reservoir capacity to the spillway crest is 9192 AF, with 370 AF assigned to dead storage. Total storage capacity to first overtopping dam crest elevation (111.4 feet, local datum) is 14,170 AF. The dam has a hydraulic height of 59 feet. On the basis of criteria in the U. S. Army Corps of Engineers Recommended Guidelines for Safety Inspection of Dams (Ref. 1), the project is intermediate in size. There are a sufficient number of inhabitable structures in the downstream flood plain that a dam failure could endanger many lives and cause extensive property damage. The downstream hazard potential is therefore high (Category 1). However, no dam breach analysis or routing of a dam breach flood was made for the downstream area and the conclusions on probable damage are based on a brief field inspection and engineering judgment.

The guidelines recommend that the spillway design flood (SDF) for an intermediate-size, high downstream hazard potential dam be the Probable Maximum Flood (PMF). The PMF is the flood expected from the most severe combination of meteorologic and hydrologic conditions that are reasonably possible in the region. Routing of the estimated PMF (excluding any runoff contributions upstream of the Yellow Water Storage Project) developed for the safety evaluation of Petrolia Dam showed that the dam is capable of controlling a flood

having ordinates approximately equal to 2 percent of the PMF hydrograph ordinates. The embankment materials are classified as being moderately erodible and can be expected to remain stable only for a short period of time under an overtopping condition. A flood event greater than 2 percent of the PMF ordinates results in overtopping and failure of the dam. Also, a breach of the Yellow Water Storage Project could induce failure of Petrolia Dam.

Several seepage areas were identified during the field investigation, and others reviewed as reported in previous inspection reports. None of the seepage areas examined suggest that a critical condition exists. However, the inspection took place during a low pool period. Monitoring of these seepage areas is required during periods of sustained high reservoir pool levels. The embankment shows no signs of instability; however, there is insufficient information presently available to fully evaluate stability.

A comparison of report findings with inspection guidelines shows Petrolia Dam to have insufficient storage and/or spillway discharge capacity to safely handle one-half of the PMF and therefore the spillway is seriously inadequate and the dam is considered unsafe.

RECOMMENDATIONS

Immediately develop, implement, and test a downstream warning plan. Consolidate the Petrolia warning plan with the plan developed for the Yellow Water Storage Project. Perform the recommended studies and repairs to bring the spillway and outlet works into conformance with inspection guidelines. Repair and replace the riprap on the upstream face of the dam, in the outlet works stilling basin, and in the spillway stilling basin. Using suitable material, fill and properly compact all depressions along the dam crest. Install additional piezometers and establish a regular monitoring program to evaluate underseepage and abutment seepage, with special emphasis during periods of high reservoir levels. Repair the erosion gullies on the downstream dam embankment face.

Conduct more detailed hydrologic and hydraulic routing studies to better determine the downstream hazard and required spillway capacity. Take into account the upstream Yellow Water Reservoir in the studies. Conduct geotechnical engineering studies to evaluate embankment stability and void spaces below the spillway floor. Modify the project as studies indicate.

Monitor the bank erosion and sloughing on the left stilling basin wall of the spillway and perform the necessary rehabilitation. Determine whether the spillway crest cutoff wall contacts bedrock and if water can pass under the cutoff. Conduct periodic inspections of the project at 5-year intervals by engineers experienced in dam design and construction. Develop and implement a periodic maintenance plan for the dam and appurtenant structures.

Prior to performing engineering studies and remedial construction, the project operator should coordinate the work with the Montana DNRC, Dam Safety Section, to insure compliance with all pertinent laws and regulations.

Mike Keene

Michael D. Keene, P.E.

MICHAEL D.
KEENE
4197 E

PERTINENT DATA SUMMARY

1. General

Federal ID Number	MT 00008
Landowner and Project Administrator	Montana Department of Natural Resources and Conservation
Operator	Petrolia Water Users' Association
Purpose	Primarily irrigation, with some recreation, sedimentation, and flood control
Location	Sections 25 & 36, T14N, R27E, MPM
County, State	Petroleum County, Montana
Watershed	Flatwillow Creek (tributary to the Musselshell River)
Hazard Potential	Category 1 (High)

2. Reservoir

Surface Area at Spillway Crest Elevation 102.0 feet	515 acres
Dead Storage to Elevation 70.0 feet	370 acre-feet
Active Storage to Spillway Crest Elevation 102.0 feet	9192 acre-feet
Active Storage to First Overtopping Dam Crest Elevation 111.4 feet	13,800 acre-feet
Total Storage to First Overtopping Dam Crest Elevation 111.4 feet	14,170 acre-feet
Active Storage to Dam Crest Control Elevation 112.0 feet	13,966 acre-feet
Total Drainage Area	612 square miles
Drainage Area Excluding Yellow Water Reservoir Drainage Basin	557 square miles

PERTINENT DATA SUMMARY
(Continued)

Reservoir Water Surface Elevation on the Day of the Inspection	94.4 feet

3. Spillway

Crest Elevation	102.0 feet
Type	Ogee crest with concrete chute
Crest Width	100 feet
Chute Width	100 feet
Spillway Capacity (to First Overtopping Dam Crest Elevation 111.4 feet)	10,200 cfs

4. Outlet Works

Gates	One 60-inch x 60-inch slidegate valve, two 60-inch diameter slidegate valves, and one 30-inch diameter slidegate valve
Control	Manual operators
Conduit	330 feet of 60-inch diameter concrete pipe and 105 feet of 30-inch diameter concrete pipe
Capacity	
To Spillway Crest	425 cfs
To First Overtopping Dam Crest Elevation 111.4 feet	470 cfs

5. Dam

Type	Zoned earth fill
Structural Height	76 feet
Hydraulic Height	59 feet
Design Crest Control Elevation	112.0 feet
Existing First Overtopping Dam Crest Elevation	111.4 feet
Total Crest Length	3635 feet
Dam	961 feet
Dike	2574 feet
Spillway	100 feet
Design Crest Width	20.5 feet
Existing Crest Width	Variable: 12 to 17 feet
Upstream Slope	1V on 4H, 1V on 3H, 1V on 2H
Downstream Slope	1V on 2H

Note: All project elevations are relative to a local survey datum.

CHAPTER 1
BACKGROUND

1.1 INTRODUCTION

1.1.1 Authority

This report summarizes the Phase I inspection and evaluation of Petrolia Dam. The project is owned and administered by the Montana Department of Natural Resources and Conservation (DNRC), and operated and maintained by the Petrolia Water Users' Association. Reservoir lands are deeded to the State of Montana.

The National Dam Inspection Act, Public Law 92-367 dated August 8, 1972, authorized the Secretary of the Army, through the Corps of Engineers, to conduct safety inspections of non-federal dams throughout the United States. Pursuant to that authority, the Chief of Engineers issued "Recommended Guidelines for Safety Inspection of Dams" in Appendix D, Volume 1 of the U.S. Army Corps of Engineers' Report to the United States Congress on "National Program of Inspection of Dams" in May 1975.

The recommended guidelines were prepared with the help of engineers and scientists highly experienced in dam safety from many federal and state agencies, professional engineering organizations and private engineering consulting firms. Consequently, the evaluation criteria presented in the guidelines represent the comprehensive consensus of the engineering community.

Where necessary the guidelines recommend a two-phased study procedure for investigating and evaluating existing dam conditions so deficiencies and hazardous conditions can be readily identified and corrected. The Phase I study is:

(1) a limited investigation to assess the general safety condition of the dam

(2) based upon an evaluation of the available data and a visual inspection

(3) performed to determine if any needed emergency measures and/or if additional studies, investigations and analyses are necessary or warranted

(4) not intended to include extensive explorations, analysis or to provide detailed alternative correction recommendations.

The Phase II investigation includes all additional studies necessary to evaluate the safety of the dam. Included in Phase II, as required, should be additional visual inspections, measurements, foundation exploration and testing, material testing, hydraulic and hydrologic analyses and structural stability analyses.

The authority for the Corps of Engineers to participate in the inspection of non-federally owned dams is limited to Phase I investigations with the exception of situations of extreme emergency. In these cases the Corps may proceed with Phase II studies but only to the extent needed to answer serious questions relating to dam safety that cannot be answered otherwise. The two phases of investigations outlined above are intended only to evaluate project safety and do not encompass in scope the engineering required to perform design or corrective modification work. Recommendations contained in this report may be for either Phase II safety analyses or detailed design study for corrective work.

The responsibility for implementation of these Phase I recommendations rests with the State of Montana, Department of Natural Resources and Conservation. The operator is urged to contact the Montana DNRC prior to taking any action on report recommendations. It should be noted that nothing contained in the National Dam Inspection Act, and action or failure to act under this Act, shall be construed (1) to create liability in the United States or its officers or employees for the recovery of damage caused by such action or failure to act or (2) to relieve an owner or operator of a dam of the legal duties, obligations, or liabilities incident to the ownership or operation of the dam.

The investigation process allows for report review by the Montana DNRC (project owner/administrator) and the Petrolia Water Users' Association (project operator). Review comments are considered before final publication of the Phase 1 Inspection Report (See Appendix F).

1.1.2 Purpose and Inspection

The findings and recommendations in this report were based on visual inspection of the project, minimal field survey measurements, and a review of available design and operation data. The purpose of the inspection was to make a general assessment as to the structural integrity and operational adequacy of the dam embankment and its appurtenant structures.

Inspection procedures and criteria were those established by the Recommended Guidelines for Safety Inspection of Dams (Ref. 1).

The visual inspection of Petrolia Dam was made on September 26, 1979. HKM Associates personnel who attended the field inspection and contributed to this report were:

Dan Dyer, Geotechnical Engineer
Mike Keene, Hydraulics/Hydrology, Team Leader
Dan Nebel, Geology

The project was also inspected by Mr. Al Kersich (HKM Engineer and Principal-in-Charge) on September 30, 1979.

Other HKM personnel contributing to the report but not attending a field inspection were:

Dale R. Cunnington, Structural Engineer
Gary Elwell, Hydraulics/Hydrology

Other personnel present during the September 26, 1979 inspection included:

Glen McDonald, Supervisor, Montana DNRC, Dam Safety Section
Art Taylor, Dam Safety Engineer, Montana DNRC, Dam Safety Section
Jerry Bohn, Board Member - Petrolia Water Users' Association
Bill Solf, Board Member - Petrolia Water Users' Association
Kenneth Welter, Board Member - Petrolia Water Users' Association

Inspections of Petrolia Dam have historically been performed by the Montana DNRC, with an occasional inspection by the Soil Conservation Service (SCS). One or several representatives of the Petrolia Water Users' Association are generally present during the inspections (Ref. 13).

1.2 DESCRIPTION OF PROJECT

1.2.1 General

Petrolia Dam is a zoned earth fill dam located in the S½ of Section 25 and N½ of Section 36, T14N, R27E, M.P.M., Petroleum County, Montana (Appendix A and Ref. 2, 3, 4, 5).

The dam and reservoir form a multi-purpose facility within the Missouri River Basin by containing the flows of Flatwillow Creek and Yellow Water Creek. Petrolia water is either released into an irrigation distribution system or returned directly to Flatwillow Creek. Flatwillow Creek travels

another 16 miles before joining the Musselshell River (Appendix B and Ref. 6). The nearest downstream community is Mosby, Montana, which is located on the east bank of the Musselshell River approximately 21 miles northeast of the dam. There are several farms/ranches located in the flood plain of Flatwillow Creek and the Musselshell River.

The dam is constructed on property deeded to the State of Montana, with a total of 515 acres deeded for the reservoir pool right-of-way. The Petrolia Water Users' Association operates and maintains the project. The State of Montana administers the project, and provides technical and financial assistance (Ref. 5, 7).

Petrolia Dam has a hydraulic height of 59 feet and impounds 13,800 AF (active storage only) at the first overtopping dam crest elevation (111.4 feet). Based on a visual reconnaissance and engineering judgment, approximately 3 farms/ranches, as well as private roads, bridges, irrigation distribution systems, and agricultural land will be affected by a sudden breach of the dam. On the basis of this information and in accordance with the Recommended Guidelines (Ref. 1), the project is classified intermediate in size and the downstream hazard potential is high (Category 1).

Petrolia Dam was constructed primarily to provide storage and regulation in support of irrigation practices. Incidental capacity is provided for sediment accumulation and floodwater detention. Some recreational benefits have also been associated with the Petrolia project. Project designers assigned 370 AF to dead storage. Active storage to the normal pool level, or the spillway crest (elevation 102.0 feet), is 9192 AF (Ref. 4, 5). An additional 4608 AF are available for flood surcharge storage between the spillway crest and the first overtopping dam crest elevation.

Petrolia Dam has a total upstream contributory drainage area of 612 square miles. A small portion of the total watershed is controlled by Yellow Water Reservoir; in particular, the Yellow Water Storage Project controls runoff from a 55-square mile drainage basin. (Yellow Water Reservoir is located approximately 13 stream miles upstream of Petrolia Dam on Yellow Water Creek). The Petrolia watershed is characterized by both mountainous and prairie drainages. Elevations in the basin range from 8730 feet msl at Greathouse Peak in the Snowy Mountains to about 2900 feet msl at the reservoir (Ref. 2, 6). The reservoir is located in open country with some scattering of large cottonwood trees along the river bottom. The terrain bordering the reservoir is generally flat to gently rolling (Photo 1 of Appendix C).

1.2.2 Regional Geology

Petrolia Dam is situated in the Great Plains province on Flatwillow Creek in central Montana. Local topography consists of high buttes and broad undulating plains broken by numerous gullies and valleys. Structurally, the dam is located near the axis of a broad anticline in a transition area between two large structural provinces: the Blood Creek syncline-Cat Creek anticline to the north and east, and the Snowy Mountain uplift to the west. The strata in the area dip gently to the east with the Eagle Sandstone forming caprock escarpments on the surrounding buttes and soft shale beds of the Colorado group forming the lower slopes and surrounding plains. No faults were noted in the area and none have been noted by previous investigators.

1.2.3 Seismicity

Petrolia Dam is in a relatively quiet seismic zone with the majority of the region's seismic events occurring in the southwestern Montana-Yellowstone Park area. Since 1925, Montana has experienced five shocks that reached intensity VIII or greater (Modified Mercalli Scale). The closest epicenter occurred at Helena, Montana which is approximately 180 miles west of the damsite. No shocks of intensity IV or greater have been reported within a 100-mile radius of the site (as of January, 1974). The dam is located in Zone 1 of the Seismic Zone Map of the Contiguous States (Ref. 1) which is based on known distribution of damaging earthquakes and their associated intensities. The potential for earthquake damage in Zone 1 is minor and in general, dams in seismic Zone 1 may be assumed to present no hazard from earthquakes provided static stability conditions are satisfactory and conventional safety margins exist (Ref. 1, 8).

1.2.4 Site Geology

No drill hole information is available for the site; hence, information from surficial interpretation is used to develop this section.

The reservoir basin is formed in a sandy shale sequence with 5 to 10-foot sandstone beds forming ledges throughout the area. The valley section consists of alluvial deposits of sandy gravel which are estimated to be 20 feet or less in depth. No large areas of slope instability were noted in the reservoir basin, which is generally expected because of the gentle slopes in the basin.

1.2.5 Sedimentation

The catchment area above Petrolia Reservoir consists primarily of natural grass-covered range land, subirrigated and irrigated lands, and dense forest cover in the mountain areas. Siltation did not appear to be a critical factor in the original design, with a total allocation of 370 AF to dead storage. There have been no sediment studies performed at Petrolia Reservoir or in the upstream watershed. Field investigations and operation and maintenance reporting have not indicated major concerns or problems relative to sedimentation.

1.2.6 Design and Construction History

The Petrolia Storage Project was designed and constructed by the Montana State Water Conservation Board (SWCB). Construction was financed with SWCB funds. The eventual project operator, Petrolia Water Users' Association, was incorporated on April 21, 1950 for a period of forty years. The construction contract for Petrolia Dam was awarded to Haggerty and Messmer on June 1, 1950, with project completion dated August 2, 1951 (Ref. 9). The first water was released into the irrigation ditches in 1952.

Some soil sampling and testing was accomplished on the Petrolia Storage Project. Tests included: moisture-density relationships, specific gravity, grain-size distribution, and permeability. These tests were performed to establish physical and engineering properties of the on-site soils. The moisture-density relationships were performed to establish a basis for density control, but it is not known if a design density was selected or whether density control tests were performed during construction (Ref. 10, 11).

Since completion of the original construction work, the Montana DNRC and the local water users' assocation have performed some repair and improvement work. In late 1952 and early 1953, holes were drilled and pressure grouted along the dam to reduce seepage. The grout program was apparently successful in stopping the majority of the seepage (Ref. 12). Riprap along the upstream face of the dam was rearranged and/or replaced in November and December of 1964 (Ref. 7, 9). In November and December of 1968, the concrete floor of the spillway was repaired and the spillway return channel was cleared and enlarged (Ref. 7). Similar spillway work was performed in 1970 (Ref. 4,5). Details of the conditions prior to these repairs are not available. A SCS interoffice memorandum dated June 13, 1975 recommended an exploratory drilling program be performed upstream of the spillway. The purpose of the drilling program was to determine if the concrete spillway is founded on bedrock (Ref. 7 and

13). The project files, and personal contacts with the water users and DNRC engineers, indicate that this drilling program was never performed.

Three piezometer holes were drilled and monitoring tubes installed in 1975 along the downstream face (Ref. 13 and Exhibit D3 of Appendix D). One year later three more piezometer tubes were added (Exhibit D3). In the fall of 1977, minor patchwork repair was made to the outlet works conduit immediately downstream of the emergency and operating valves. A drain trench and pipe were installed in the left abutment in the spring of 1978 (Ref. 13).

CHAPTER 2
INSPECTION AND RECORDS EVALUATION

2.1 HYDRAULICS AND STRUCTURES

2.1.1 Spillway

The spillway for Petrolia Dam is located on the right (south) side through the earthen dike section of the dam. The spillway is a concrete lined chute having a gravel approach section and a concrete ogee crest. The approach channel is approximately trapezoidal in shape with a minimum base width of 100 feet. The side slopes are nearly vertical, particularly near the spillway crest. The approach channel is constructed on approximately a horizontal grade, and transitions to the reservoir basin about 125 feet upstream of the spillway crest. The ogee crest was set at elevation 102.0 feet (local datum). There are three concrete piers located on 25-foot centers across the ogee crest. The piers support a footbridge over the spillway. Spillway width at the ogee crest and in the chute is 100 feet. Energy dissipation is provided by a flip bucket at the end of the chute, and a stilling basin constructed in rock (Sheets 4 and 5 of Exhibit D1). There is a 29-foot drop from the spillway crest to the design stilling basin invert. Spillway flows pass from the stilling basin into Johnson Coulee, and travel as close as 200 feet to the dam embankment before returning to Flatwillow Creek. The encroachment to within 200 feet of the embankment has not been historically reported as a concern, and the September 1979 field inspection verified that there is no immediate concern. Pertinent construction plans are provided in Exhibit D1 of Appendix D.

Original hydraulic rating information for spillway operation was provided by the Montana DNRC (Ref. 14). Adjustments were made in the original calculations to reflect the constructed pier arrangement on the ogee crest and to better represent existing conditions (Exhibit D1 and Ref. 15). This new rating was developed by using the weir head-discharge relationship with the head-dependent discharge coefficient varying from 2.99 to 3.76. Head losses in the approach channel were assumed to be negligible. Spillway capacity to the first overtopping dam crest elevation (111.4 feet) is estimated to be 10,200 cubic feet per second (cfs). Spillway rating data is provided in Exhibits E3 and E4 of Appendix E.

The spillway approach section is quite irregular along the flowline due to aggradation and degradation of the gravel and cobble material (Photos 33 and 34 of Appendix C). The approach channel banks have been eroded and are near vertical, particularly near the concrete headwalls of the spillway chute. Regrading and replenishment of the material

is suggested. A hole exists immediately upstream of the concrete ogee crest (Photos 33 and 34). This particular hole may be the result of local scour activity, or more importantly, it may be that the surface material has subsided due to water and subsurface material movement under the spillway cutoff wall. It is not obvious from the project drawings and design files as to whether the spillway cutoff wall was keyed to bedrock; in fact, more evidence supports the absence of "keying" and the possibility of substantial water movement beneath the spillway floor. There are some void spaces (24 to 36 inches deep) located beneath the concrete floor of the spillway near the change-in-slope point of the chute. A few spot measurements were taken, but not enough to confirm the total extent of this void. The project operator indicates that this same problem became evident a few years ago, and it was necessary to fill the void space with large gravel aggregate. It was apparent during the field inspection of September 1979 that the previous work did not overcome the problem.

With the exception of the ogee crest and piers, the concrete of the structure, particularly in the vicinity of the slab joints, shows signs of deterioration. Concrete patchwork has been performed periodically on the spillway walls and floor (Ref. 4, 5, 7, 13). Rehabilitation work is again required on the walls and floor (Photo 35 of Appendix C). Concrete deterioration was also noted on the spillway flip bucket at the exit (Photo 39). In addition, water continuously sets in the flip area which accelerates damage to the local concrete during repeated freeze-thaw cycles (Photo 39). An appropriate relief is needed to eliminate the opportunity for ponding water.

The stilling basin and return channel have been enlarged and cleaned on several occasions in the past (Ref. 7). Rather severe undercutting has occurred at the spillway exit as shown in Photo 39 of Appendix C. This undercutting action is not considered to be critically progressing because the erosion has basically reached nonerosive material. The basin and channel banks continue to experience sloughing and erosive cutting (Photos 19, 36, 37, and 38). Most of the bank deterioration is attributed to spillway flows; however, seepage daylights along the left stilling basin wall and creates a slightly unstable condition (Photo 36). None of the erosion is presently threatening the dam.

2.1.2 Outlet Works

The outlet works for Petrolia Dam is located about 100 feet from the left abutment contact and aligned approximately along the toe of the left valley wall. The outlet facility

consists of: an approach channel; a concrete intake structure; a 60-inch diameter conduit; two wet-well chambers (a control tower which contains the emergency and operating gates and an outlet for the high-line canal, and a second wet-well chamber containing a vertical turbine pump and a control gate); delivery pipelines to the high-line canal; concrete outlet structures (one at the high-line canal outlet, one at the low-line canal outlet, and one at the return to Flatwillow Creek); a stilling basin; and a return channel. Construction plans are provided in Exhibit D1 of Appendix D.

An approach channel was constructed from the natural channel of Flatwillow Creek to the concrete intake structure. The intake was designed as a flared inlet with a metal trash rack. Flowline elevation for the intake was set at 53.0 feet, which is 17 feet below the assigned dead storage elevation (Ref. 4, 5, and Sheet 2 of Exhibit D1). The intake structure could not be inspected due to the height of the reservoir pool at the time of inspection.

The outlet conduit is a 60-inch diameter reinforced concrete pipe. Total conduit length from the intake structure to the outlet structure at the return to Flatwillow Creek is about 330 feet. The conduit was inspected only from the emergency gate downstream to the outlet. Minor deterioration of the conduit wall was evident immediately downstream of the control tower operating gate (at approximately Station 2+07 as shown on Sheet 2 of Exhibit D1) in a region that had been previously repaired (Photos 24 and 25 of Appendix C and Ref. 13). A small leak was observed at a conduit joint one barrel section upstream of the outlet to Flatwillow Creek. The joint-filler material at conduit joints between the second wet-well ("second" as numbered in the downstream direction) and the outlet is rather irregular and occasionally absent (Photo 26).

The control tower (first wet-well as numbered in the downstream direction) is located 7 feet downstream of Petrolia Dam centerline. The tower contains a 60-inch by 60-inch rectangular slidegate (emergency valve), a 60-inch diameter slidegate (operating valve), and a 30-inch slidegate and outlet for the high-line irrigation canal. The 30-inch gate is set on the north wall of the control tower. Water to the high-line system cannot be delivered until the reservoir pool reaches an elevation greater than 91.5 feet. A control house sets over the chamber and gate operators (Photos 4 and 20 of Appendix C and Sheets 2 and 3 of Exhibit D1). All gate operators are located on the top floor level of the control tower, which is approximately coincident with the dam crest elevation. The following specifications were obtained from the inspection report files (Ref. 13):

2 Armco Hardesty Pedestal Lifts
2 Gates V34x1
Stems 3-1/2"; crank shafts 1-3/4"
1 Slidegate No. 13-30
Stem 1-1/8"

For inspection purposes, the emergency gate was closed and the operating gate in the control tower was opened. The conduit downstream of the control tower was partially pumped/ drained. Complete "seating" was not possible for the emergency gate as shown in Photo 21 of Appendix C. Some metal corrosion was observed on the emergency and operating gates, but not considered serious or critical at this time (Photos 21 and 22). Erosion and cavitation damages were evident along the metal seating ring for the operating gate in the control tower (Photos 24 and 25). The delivery pipeline from the control tower to the outlet to the high-line canal is a 30-inch precast concrete pipe with lock joints, and is approximately 105 feet long. ,The outlet structure located at the entrance to the high-line canal is a reinforced concrete structure with flared sidewalls and vertical headwall (Sheet 3 of Exhibit D1). The 30-inch pipe was not inspected. The outlet structure, and the canal in the immediate vicinity of the dam embankment, was in reasonably good condition. It appears unlikely that the capacity of the high-line canal would be exceeded even if the 30-inch gate is in the fully open position and the reservoir is at high pool levels. There is a remote possibility during high pool levels that the 30-inch gate could be fully open and one of the control gates along the 60-inch conduit restricting flow to such an extent that high discharge rates would be forced through the high-line system. Also, there is an extremely remote chance that inflow through the 30-inch pipe would be combined with pump contributions to result in excedence of canal capacity. Even if one of these conditions existed, canal bank overtopping probably would not result in a complete breach as the high-line canal is a cut section near the dam. Only after a prolonged saturated condition might an earth slide or a breach occur. Canal bank overtopping or a breach should not prove critical to the structural integrity of the dam embankment unless this condition lasts for a considerable length of time.

The second wet-well chamber is located approximately 85 feet downstream of the dam centerline and about 90 feet upstream of the outlet to Flatwillow Creek. This chamber houses a vertical turbine pump and a 60-inch slidegate for the low-line irrigation canal (Sheet 2 of Exhibit D1). The vertical turbine pump is used primarily to provide irrigation water to the high-line canal, and possibly to the low-line canal when gravity feed is not possible. The 60-inch slidegate is mounted on the south wall of the chamber and operates across the 60-inch

conduit leading back to the creek. A control house is situated over the chamber (Photos 5 and 32 of Appendix C), and contains the 60-inch slidegate operator and the pump motor/controls. The gate and operator specifications are as follows (Ref. 13):

1 Hardesty V12x1 gate
Stem 3"
Crank Shaft 1-3/8"

The maintenance ladder was missing from the second wet-well chamber. Also, the concrete walls are deteriorating with some leakage (Photos 27 and 28). The slidegate has experienced a considerable amount of corrosion. The gate support ribbing is weakening from corrosion and the metal is flaking (Photo 23).

Water can enter the low-line irrigation canal by gravity feed if the reservoir pool is at an elevation greater than 70.0 feet. At lower pool elevations, pumping would be necessary to provide water supply to the low-line canal.

The outlet to the low-line canal is a reinforced concrete structure with flared sidewalls. Some concrete deterioration was evident on the outlet structure. The canal is irregularly shaped along the flowline, but in reasonably good hydraulic and structural condition. With improper operating procedures and with high pool levels, it appears possible that the low-line canal capacity could be exceeded. The structural integrity of the lower reach of outlet conduit and the outlet structure to Flatwillow Creek could be endangered if canal bank overtopping and breaching occurred. Major structural damage to the dam embankment is not anticipated during an overtopping/ breach condition due to the local terrain and materials in the embankment toe.

Flow not diverted or pumped into the irrigation distribution system continues to pass through the 5-foot diameter outlet works conduit and is returned to Flatwillow Creek. These releases transition from the conduit to the natural channel by passing through an outlet structure, a stilling basin, and a short reach of return channel (Sheets 1 and 2 of Exhibit D1). The outlet structure is a reinforced concrete flared section with a headwall and cutoff wall (Photo 30 of Appendix C). The outlet was set at a flowline elevation of 52.0 feet (Exhibit D1 of Appendix D). Concrete surfaces show a small amount of pitting, probably due to seasonal freeze-thaw activity. Severe undercutting and rock displacement have occurred at the outlet to the stilling basin (Photo 29). The erosion and deterioration are not critical to the safety of the project, but corrective measures are needed.

The stilling basin consists of a depressed channel section (or scour hole), an armor plate from the depressed section to a raised channel invert, and sloping channel banks. All features include a rock riprap cover (Photo 30 of Appendix C). Undercutting has occurred at the outlet structure (as described above for the outlet structure), rock has been displaced from the armor plate, and some riprap subsidence has been experienced along the channel banks (Photos 29 and 30).

A short reach of return channel completes the transition to Flatwillow Creek. Very little definition remains as to design channel geometry and grade, primarily because of bank sloughing and bed material displacement (Photos 18 and 31).

There is potential for pressure pipe flow downstream of the first wet well. Based on field inspection and project data review, problems resulting from pressure flow are not anticipated.

Outlet works capacity with the reservoir at the spillway crest (elevation 102.0 feet) is estimated to be approximately 425 cfs, and at the first overtopping dam crest (elevation 111.4 feet), the capacity is 470 cfs. Estimates are based on the assumptions that all operating gates are in the fully open position, full conduit flow exists throughout the entire length, and Manning's "n" of 0.011. Detailed consideration was not given to flow distribution with respect to all outflow branches in the system. That is, for this level of study, outflow was considered only through the 60-inch conduit from the inlet to the return channel outlet. Final outlet works rating data is summarized in Exhibits E3 and E4 of Appendix E.

2.1.3 Freeboard

Flood routing (Section 2.2.3) indicates the dam overtops during the guidelines' (Ref. 1) recommended spillway design flood (PMF), and therefore, no freeboard exists for such conditions. The vertical distance from the spillway crest (elevation 102.0 feet) to the first overtopping dam crest elevation (111.4 feet) is 9.4 feet. Available records indicate that probable historic highwater levels for Petrolia Reservoir occurred in May 1964, June 1975, and March 1979. In all cases, it was estimated that the flow depth over the spillway crest was about 2.0 feet, which is equivalent to a pool elevation of 104.0 feet. The existing normal highwater and debris lines substantiate the probable historic highwater level (Photos 6 and 7 of Appendix C). The vertical distance between the reservoir pool and the first overtopping elevation at the time of the September 1979 field inspection was 17.0 feet.

Petrolia Reservoir is oriented in an east-west direction, with the dam on the east side. The prevailing wind for this region is identified as being westerly (Ref. 16, 17). The reservoir location and orientation can be observed in Appendices A and B. The effective fetch length is calculated to be 2 miles. Hence, the minimum freeboard allowance should be about 5 feet (Ref. 15). This 5-foot minimum allowance does not apply to an event as severe as the PMF where the design objective would be to provide only enough freeboard to maintain structural integrity of the dam. Although the dam will overtop during the PMF and lesser floods, the vertical distance between normal pool and the dam crest is sufficient to prevent overtopping by wind-generated waves.

2.2 HYDROLOGY

2.2.1 Physiography and Climatology

The 612-square mile catchment area above Petrolia Reservoir is basically rectangular in shape, with the length dimension being considerably greater than the width. In particular, the drainage area is approximately 60 miles long and only 15 miles wide at its widest point. The topography includes flat plains, rolling prairies, foothills, and mountainous areas. The immediate vicinity of the reservoir can primarily be characterized as plains topography. The watershed rises from the reservoir in a westerly direction by gradual slopes and bench lands to the higher mountainous elevations of the Snowy Mountains. Petrolia Dam backs up water on both Flatwillow Creek and Yellow Water Creek, thus forming the two major arms of the reservoir.

Soils along the stream bottoms are generally sandy loams and gravels. Bench-land soils commonly range from clay loams to silty clay loams, becoming gravelly with depth (Ref. 9, 18). The regional climate is classified as distinctly continental, and characterized by abundant sunshine, low relative humidity, light rainfall, and wide daily and seasonal variations in temperature. However, the regional climate does not have the extreme variable pattern common to the more mountainous western sections of Montana. In general, the valleys are relatively dry during the colder months and wet during the late spring and early summer. The wettest part of the year in the mountains is generally from midwinter to early spring. Snowfall in the prairie sections of the Petrolia watershed is generally light and does not tend to remain on the ground for long periods of time. It is not uncommon for the region to experience winter warming spells with associated thawing temperatures. Precipitation in the Petrolia watershed ranges from about 13.5 inches at Flatwillow (10 miles southwest of the dam) to amounts in excess of 50 inches in the

Big Snowy Mountains. The average annual temperature at Flatwillow is approximately 45°F. Winters are typically cold, with January being the coldest month. The monthly average temperature for January at Flatwillow is about 20°F. During the summer, July is typically the warmest month with an average temperature of about 70°F (Ref. 16, 17, 18).

The U.S. Geological Survey accumulated streamflow data on Flatwillow Creek, the main tributary to Petrolia Reservoir, at two different locations in the past. Flatwillow Creek near Flatwillow, Montana (gage number 06127900) has the longest record of the two, with 41 years of peak flow measurements. Drainage area above this particular gage is 188 square miles. The historic peak flow occurred in June 1917 when 954 cfs passed the gaging station. To emphasize the peak runoff pattern, 11 of the 41 annual peak events occurred in May and 18 occurred in June (Ref. 19).

The other stream gage maintained by the U. S. Geological Survey is identified as Flatwillow Creek near Winnett, Montana (gage number 06128200). This gage was located immediately downstream from Petrolia Dam and had an upstream drainage area of 642 square miles. The period of record for peak discharge measurements is as follows: 1923 - 1925, 1927 - 1929, 1931 - 1932, 1946, 1950 - 1951. Most of these annual peak events occurred in June (Ref. 19).

A regular monitoring program of the Petrolia Reservoir water surface is not maintained. Occasional measurements of the reservoir contents are made by a measuring tape in the outlet works control tower. Available records indicate that peak water levels have occurred in May 1964, June 1975, and March 1979. This is not to imply that high reservoir levels are exactly concurrent with relatively high runoff events from the contributory watershed, basically because seasonal reservoir operation must also be considered. However, because Petrolia Reservoir has a relatively small usable storage content, it can generally be implied that high reservoir pools are the result of large flood events and/or seasonally high watershed yields.

2.2.2 Estimated Probable Maximum Flood (PMF)

The inspection guidelines (Ref. 1) recommend that the spillway design flood (SDF) for a project of this type be the probable maximum flood (PMF). The PMF is the flood that may be expected from the most severe combination of critical meteorologic and hydrologic conditions that are reasonably possible in the study region. The probable maximum precipitation (PMP) and the estimated PMF were developed for the Petrolia drainage basin, exclusive of the drainage area

upstream of Yellow Water Reservoir. It was assumed for the Phase 1 investigation that the Yellow Water Storage Project could contain all runoff being generated from its contributory area concurrent with the PMP event on other parts of the Petrolia watershed. Specific to Petrolia Reservoir, the ratio of the reservoir area to the non-reservoir area is less than 1 percent; therefore, the two areas were not separated for the purpose of this analysis.

Petrolia Reservoir is located west of the 105th meridian and east of the continental divide, hence PMP values were calculated using procedures contained in the National Weather Service Technical Paper 38 (TP-38) (Ref. 20). TP-38 provides PMP values for durations to 24 hours, with extensions to 48-hour and 72-hour durations obtained by applying a factor of 110 percent and 115 percent, respectively. An extrapolation of TP-38's area-reduction curve was required because the drainage area above Petrolia Reservoir (excluding the drainage area above Yellow Water Reservoir) exceeds 400 square miles. The curve extrapolation was performed by the Corps of Engineers, Seattle District. After applying the suggested areal reduction factors for the 557-square mile drainage basin, the PMP 1-hour, 6-hour, 12-hour and 24-hour precipitation values were 5.5 inches, 12.2 inches, 15.0 inches and 18.1 inches, respectively. The 48-hour precipitation was determined to be 19.9 inches, and the 72-hour precipitation was 20.8 inches.

The 6-hour increments for the total 72-hour storm were arranged in a critical distribution using criteria presented in the National Weather Service Hydrometeorological Report No. 43 (Ref. 21). In particular, the 6-hour rainfall increments were arranged according to pattern "e". Further subdivision of the calculated rainfall increments was required to provide compatibility with the duration of the unit hydrograph. A 30-minute unit hydrograph was selected for Petrolia Reservoir using criteria presented in the SCS Hydrology Handbook (Ref. 22). The unit hydrograph was developed for the Petrolia basin using the U.S. Army Corps of Engineers' computer program HEC-1 and the SCS method (Ref. 23). The PMP storm was plotted in the form of a depth-duration curve for convenience in selecting incremental rainfall values. The 30-minute PMP values, within the peak 6-hour increment, were ordered according to the reverse pattern of the largest 12 unit hydrograph ordinates.

Rainfall losses were assumed equal to zero due to ground conditions. Hence, excess rainfall was assumed equal to the rainfall amounts.

The runoff condition, or PMF, resulting from a PMP storm was estimated using the PMP values and the unit hydrograph approach. The resultant PMF has a total runoff volume of 616,700 AF.

2.2.3 Flood Routing

The PMF resulting from the PMP rainfall/runoff event was routed through Petrolia Reservoir using the computer program HEC-1 (Ref. 23). Runoff from an antecedent storm was not specifically considered; however, it appears reasonable to assume the initial reservoir level prior to routing the PMF is at the spillway crest (elevation 102.0 feet, local datum). The support rationale for this assumption is that Petrolia Reservoir characteristically has a small reservoir pool and is not capable of totally absorbing rather severe flood events. Rapid drawdown of the reservoir pool can be accomplished with the 60-inch outlet conduit. However, it is not likely that the project would be operated in this manner unless it was an emergency measure. The water users would prefer to store as much water as possible in preparation for the irrigation season. Also to be considered in releasing high discharge rates through the outlet works is the probable damage that would result in the outlet stilling basin and return channel. This damage would probably remain localized and not threaten the dam embankment. In summary, the "sense of emergency" is not likely to be realized by the project operators to the extent storage water and downstream conditions would be sacrificed in anticipation of a PMF event.

Reservoir area-capacity-elevation data was obtained from the DNRC files (Ref. 4, 5, 14), and is provided in Exhibits E1 and E2 of Appendix E. Petrolia discharge rating data is shown in Exhibits E3 and E4. Flow through the outlet works was included in the flood routing.

For the purposes of flood routing and according to Phase 1 investigation criteria, the dam crest elevation is the minimum elevation to which the reservoir must rise before overtopping the dam. This criteria assumes overtopping and failure of embankment-type dams to be coincidental. Based upon the embankment profile survey dated October 16, 1979 (Exhibit D2), the existing first overtopping dam crest elevation is 111.4 feet.

Flood routing showed that the dam would first overtop during the PMF when approximately 1 percent of the total PMF volume enters the reservoir. Percentages were taken of the PMF ordinates to determine what event Petrolia Dam can safely control without failure. It was found that the dam could store and hydraulically pass an event approximately equal to 2 percent of the PMF (2 percent of the PMF hydrograph ordinates).

This estimate does not include the effects of discharges from the upstream Yellow Water Reservoir during the PMF. As described above, Petrolia Dam can hydraulically pass events less than or equal to 2 percent of the PMF, but the spillway may not be able to maintain structural integrity under this type of loading in the absence of adequate support under the spillway floor. Also, substantial damage is likely to result in the low-line irrigation canal, in the outlet stilling basin, and in the return channel.

2.3 GEOTECHNICAL EVALUATION

The geotechnical evaluation of Petrolia Dam included a field investigation, and a search and review of project design data. The field inspection consisted of photo documentation, a dam crest profile survey, slope stability observations of the dam embankment, seepage observations, and measurements of the slope angles. Inspection photos are included in Appendix C, some of the construction plans are included in Exhibit D1 of Appendix D, and the crest profile survey is shown in Exhibit D2. Results of the slope angle measurements are provided in Exhibit D3.

2.3.1 Dam Embankment

Petrolia Dam is a zoned, earth fill structure which was completed in 1951. The dam has a maximum structural height above the deepest point on the foundation surface of 76 feet and a total crest length of 3635 feet. The project consists of a main dam embankment, 961 feet long between dam abutments, and a zoned earth fill dike that extends 2674 feet beyond the right abutment. Included in the dike section is a 100-foot-wide concrete spillway. The dam and dike crest width was designed to be 20.5 feet, but field measurements indicate a variable width of between 12 and 17 feet (Photos 4 and 5 of Appendix C). This is primarily the result of the riprap moving down the upstream slope, causing a narrowing of the crest. The construction plans (See Exhibit D1, Sheet 1) indicate the upstream face has a slope of 1V (vertical) on 2H (horizontal) down to elevation 102.0 feet (local datum), which is the crest elevation of the spillway, then flattens to a slope of 1V on 3H until reaching elevation 92.0 feet. Slope measurements in the field show this portion of the slope to be slightly steeper (near 1V on 2.5H). The construction plans indicate that below elevation 92.0 feet the embankment has a slope of 1V on 4H. The upstream face is covered with loose, rock riprap and some scattered debris above elevation 92.0 feet (Photos 6-9).

The downstream face has a slope of 1V on 2H. There is a moderate to sparce cover of natural grasses, weeds, and sagebrush on the downstream slope (Photos 5, 11, 12, and 13).

Details provided on Sheet 1 of Exhibit D1 indicate the embankment cross section consists of three parts identified as impervious (core), sand and gravel (shell), and riprap. A detailed description of these materials and zones is not available in the design file.

The impervious section is located in the central region of the embankment. Plans indicate that a cutoff trench is located at the base of the impervious section on the upstream side of the main dam embankment. For the dike section, the cutoff trench is more centrally located. Construction drawings indicate that the cutoff trench extends into the underlying bedrock (Sheet 1 of Exhibit D1). In the main embankment area, this trench is shown to extend approximately 13 feet into bedrock. Only a 3-foot extension into bedrock was constructed under the dike section. Further discussion relative to the cutoff trench is provided in the next section. The sand and gravel zone forms the upstream (below the riprap) and downstream exterior faces of the embankment. Based on field observations in erosional cuts, this material is pervious, well graded, and granular with no plasticity.

2.3.2 Foundation Conditions and Seepage Control

Neither soil boring or test pit logs were available; however, there were some soil test results available for review. Tests were performed on selected samples apparently taken from the borrow areas and test holes. These tests included: moisture-density relationships, specific gravity, grain-size distribution, and permeability. It is not known whether these tests were performed during design or construction. Further, it is not known with certainty what zones of material the tests represent.

Based on available information, it appears that the main section of the existing dam embankment is resting primarily on alluvial deposits of sand, gravel, and clay up to about 10 feet thick. The core trench described in Section 2.3.1 was designed to extend through these materials into the foundation bedrock.

Bedrock is a sandy shale with laminated calcareous sandstone seams. This material is relatively permeable in the horizontal direction due to a laminated-type structure. The abutments are layered masses of interbedded sandy shale and sandstone.

Settlement

The construction plans indicate that the dam crest control was established at elevation 112.0 feet. An embankment profile was surveyed on October 16, 1979 (Exhibit D2

of Appendix D). The survey shows that the maximum differential elevation along the crest is about 1.6 feet. The points of greatest depression are generally located along the main section of the embankment. It appears the dam was designed without camber. The apparent differential settlement is probably a combination of settlement within the alluvial foundation and the embankment materials.

Settlement along the outlet conduit appears to be in the range of 6 inches to 1 foot. A visible deformation in the conduit was observed between the second wet-well chamber and the outlet (Photo 26 of Appendix C). Conduit alignment upstream of the upper wet-well chamber was not observed because it necessitates an almost complete reservoir drawdown for inspection.

In summary, a history of settlement along the dam embankment is not available. The apparent settlement to date has not caused significant structural damage. Because of the age of the dam (28 years), additional significant amounts of settlement are not anticipated.

Seepage Control

At the time of the September 1979 field investigation, Petrolia Reservoir was at a relatively low level (elevation 94.4 feet, or 7.6 feet below the spillway crest). Several seepage areas were identified during this investigation, and each will be discussed in more detail below. Discussions are also presented below for comparison with seepage areas identified in earlier inspection trips made by state inspection personnel.

Seepage areas identified by HKM during the September 1979 investigation are as follow:

1. Left Abutment. Evidence of seepage was observed at the left abutment contact between the high-line canal and pump house at the low-line canal level (Photo 32 of Appendix C). This seepage area is located near elevation 70 feet. Seepage has also been observed in this area by inspection teams in the past. Although free water was not observed during this field investigation, vegetation type and extent suggests seepage from the abutment.

   In the spring of 1978, a drain trench and pipe were installed in this area. The system was intended to collect and remove a majority of the abutment seepage; however, according to local water users, the system has a limited interception capability.

It is anticipated that water will surface in this area when the reservoir pool rises and the capacity of this small relief system is exceeded. However, observation by the local water users indicate an improvement in drainage relief since the system was installed. In addition, the water users feel that by extending the collection system down the abutment (lowering the collection points), a new system would essentially eliminate all surface seeping, even during high pool periods. However, the water users do not presently have any firm plans to perform this work.

2. Considerable seepage was identified on a steep slope located approximately 300 feet downstream from the bend in the dam embankment section (that is, approximately where the main embankment section ends and the dike section begins). The seepage surfaces on an open, steep cliff which is 40 to 50 feet high. The cliff exposes the natural geological strata above Johnson Coulee (Photo 15). It is expected that the majority of this water originates in the reservoir pool and seeps along laminations in the sandstone. A smaller amount of seepage water travels through the shale bedrock.

   A small boil was identified in the Johnson Coulee stream bottom located about 300 feet from the dam embankment and below the cliff (Photo 16). Although very small, this boil is evidence of artesian pressures associated with water moving through a deep bedrock formation. This boil is not transporting soil.

3. Seepage was observed along the left valley wall of the spillway stilling basin. The water was exiting from the exposed bedrock (Photo 36). Small amounts of seepage were also noted below the concrete sill at the bottom of the spillway (Photo 39).

Seepage areas identified by state inspection personnel (Ref. 13) are described below.

4. At the intersection of the dam and right (south) abutment near Sta 11+00 (original construction survey stationing), a small seepage area emerges when the pool level is high. At the time of this investigation, September 1979, this area was dry. Moderately dense grass in this area suggests seepage earlier this spring.

5. In the past, seepage has been observed emerging at the sandstone exposure below the dike section and about 300 feet south of the dam's right (south) abutment. At the time of this investigation, September 1979, this area was dry (Photos 13 and 14).

Six piezometer tubes were identified in the field at Petrolia Dam (Photo No. 11). The tubes are approximately located on Exhibit D3 in Appendix D. Water levels were measured in 4 of these tubes and are reported on the exhibit. It should be noted that the static water level was the highest in Tube No. 4 at 6.1 feet below the natural ground surface. Details of why the piezometers were installed are limited. However, early inspection reports (Ref. 13) indicate seepage was observed in this area.

In conclusion, the groundwater seeps are not presently considered critical and do not suggest imminent danger to the dam structure. However, recognizing that the seepage areas described above probably become more active during periods of high pool levels, an inspection and monitoring program is suggested. Frequent and timely monitoring should aid in describing changes in seepage amounts and piping occurrences at different pool levels.

2.3.3 Stability

Embankment

The slope angles measured during the field investigation are shown in Exhibit D3 of Appendix D. These angles were measured with an Abney level and should be considered approximate. Except for some loss of riprap on the upstream slope, there is no outward sign of instability or trouble evident from visual inspection of the dam embankment. However, there is no stability analysis data available to evaluate the slope design. Available design information suggests that soil strength tests were not performed and stability calculations were not made. The main embankment portion of the dam does not contain piezometers and hydrostatic conditions are unknown throughout most of the dam.

Erosion

Embankment and channel bank erosion were observed at several locations. Without corrective measures, these points of erosion may develop into stability problems.

At isolated locations on the upstream face of the embankment, the bedding material under the loose rock riprap has been eroded by wave action. The riprap has moved downslope along

the embankment leaving the slope exposed and oversteepened (Photo 6). The bedding material has apparently been eroded through the voids in the riprap. The gradation of the bedding is unknown.

Accumulations of precipitation on the dam crest and in the abutment areas have spilled over the downstream embankment face and created erosion gullies. Most of these gullies are small but deserve future monitoring (Photo 17). A rather large erosion gully has developed at the contact of the embankment and the left abutment on the downstream face. This particular gully has developed to a size of 1 foot deep by 2 feet wide. There are banks approaching the vertical at the seepage areas identified previously under Seepage Control, Item Nos. 2 and 3 (Photos 15 and 36). These banks should be protected to reduce progressive bank failure.

Previous inspection reports have included a recommendation that a dike be constructed in the Johnson Coulee stream bottom to deflect the high flows away from the cliff (vertical bank located downstream of the dike section at seepage area in Photo 15). This dike has not been constructed and is no longer needed. The local water users feel the channel has naturally changed predominant flow directions and is no longer causing significant bank erosion. The absence of the dike does not affect the structural stability of the dam.

Bank erosion and sloughing were observed in the spillway stilling basin and return channel to Johnson Coulee (Photos 36, 37, and 38). This erosion does not affect dam stability, but should be considered in future project rehabilitation work. The localized bank erosion and sloughing in the stilling basin could affect the structural integrity of the spillway chute.

The reservoir shorelines are considered to be in stable condition as no major slides or scarpments were observed. The shoreline is occasionally vertical, or near vertical, and localized sloughing occurs due to wave action and saturated conditions (Photos 1, 2, 3, and 10).

2.3.4 Rock Riprap

Rock riprap has been placed on the upstream face of the dam, and in the stilling basins for the outlet works and spillway. This rock is calcareous sandstone and is generally erosion resistant. Some weathering is evidenced by the rounding of the edges and the decrease in size of the stone below the water level (Photos 6, 7, and 8). In some locations along the dike area, the quantity of riprap appears to be deficient (Photo 9). Also, the riprap has occasionally slipped down the upstream embankment face due to erosion and wave action

(Photo 6). The riprap bedding material has apparently been eroded through the voids in the riprap causing the riprap slippage.

There is insufficient riprap protection in both stilling basins. The outlet structure has been undercut to a depth of 24 to 36 inches (Photo 29).

The sill of the spillway chute has also been undercut due to inadequate riprap (Photo 39). Both banks of this stilling basin are near vertical and show evidence of recent caving as a result of undercutting (Photos 36-38).

2.4 PROJECT OPERATIONS AND MAINTENANCE

The Petrolia Storage Project is administered by the Montana DNRC through a contractual agreement with the Petrolia Water Users' Association (Ref. 5,9). Initially, the State Water Conservation Board (SWCB) had the responsibility to construct, operate and maintain water conservation projects. In 1967, the SWCB was replaced by the Montana Water Resources Board (MWRB). The new agency shifted its emphasis from construction activities to providing engineering services for local groups and water user associations, to assisting in the maintenance of previously constructed projects, and to conducting annual maintenance inspections. Responsibility for project operation and maintenance gradually shifted from the State agencies to the local groups over a period of years. Upon reorganization of state government in 1971, the MWRB was replaced by the Department of Natural Resources and Conservation (DNRC). Accounting and engineering responsibilities were departmentalized, with the Engineering Bureau of the Water Resources Division, DNRC, delegated the responsibility to provide the services previously mentioned for the MWRB. At the present time, the Petrolia Water Users' Association performs essentially all operation and maintenance activities and the State agency serves only in a supervisory capacity (Ref. 5).

Representatives of DNRC maintain frequent contact with the water user group by attending important meetings and by scheduling regular inspections of the project. They also advise the group of maintenance, repair, operation and budgetary needs. The DNRC provides assistance and direction in determining the best repair alternatives and financing sources (Ref. 5, 7, 13).

Releases from the reservoir are generally dictated by seasonal requirements. Seasons of particular interest are the spring and early-summer runoff period, and the irrigation period. The poor condition of the spillway has not always

been respected, and suggestions have been made in the past to the project operator to carry a lower pool level into the flood season. Releases during the irrigation season are monitored and controlled according to water needs and rights. Reservoir pool carry-over from year to year also plays a part in project operation. No operations manual for the project has been prepared, and operating records are kept only on some of the components for the storage and water delivery system.

Miscellaneous maintenance work has been accomplished in the past on the dam and related structures. However, this activity has been limited due to manpower and budgetary constraints.

2.4.1 Dam

There is no formal plan for periodic maintenance of Petrolia Dam. The grouting work to reduce underseepage is the most significant enhancement or repair performed since the dam was completed in 1951. Pressure grouting was performed in 1952 and 1953, approximately 18 months after dam completion. Operation and maintenance records appear to indicate that additional work was performed relative to seepage control in 1956 (Ref. 7, 9, 13). However, a confirmation of the additional work could not be made.

Other operation and maintenance work on Petrolia Dam has included: rearrangement/replacement of riprap along the upstream face of the dam in November and December of 1964; installation of 3 piezometer tubes along the downstream face in 1975 and installation of 3 additional piezometer tubes along the downstream face in 1976 (Exhibit D3); and the construction of a drain trench and installation of a drain pipe in the left abutment in the spring of 1978 (Ref. 7, 9, 13).

2.4.2 Reservoir

Petrolia Reservoir water surface elevation is measured on an irregular basis by the local water users' association, and during project inspections. Deliveries are made by a water commissioner with the aid of measuring devices and the gate stem height on the outlet works operating valves. Maintenance and repairs have not been required in the reservoir basin with the exception of work related to the recreational facilities (Ref. 9).

2.4.3 Related Structures

The spillway is an unregulated facility. All other related structures, such as the outlet works and irrigation turnouts, are controlled by the local water commissioner.

In November and December of 1968, a Montana Water Resources Board construction crew repaired the concrete floor of the spillway, and cleared and enlarged the return channel. Similar spillway work was performed in 1970. Minor patchwork repair was performed in the outlet works conduit during the fall of 1977. This repair work was directed specifically at the area immediately downstream of the valves located in the control tower (Ref. 13). An SCS interoffice memo from June 13, 1975 recommended exploratory drilling upstream of the spillway to determine if the concrete spillway is resting on bedrock. The project files, water users, dam tender and DNRC indicate the drilling as not performed.

2.4.4 Warning System

There is no formal warning system or plan of action in the event of dam distress.

CHAPTER 3
FINDINGS AND RECOMMENDATIONS

3.1 FINDINGS

Visual inspection of the dam, supplemented by analysis of the project in accordance with the guidelines (Ref. 1) and the contract performance standards, resulted in the following findings.

3.1.1 Size, Hazard Classification, and Safety Evaluation

In accordance with the inspection guidelines (Ref. 1), Petrolia Dam is classified intermediate in size and has a high downstream hazard potential rating. The recommended spillway design flood for this project is 100 percent of the PMF. Because the project can safely handle only 2 percent of the PMF without overtopping and causing the dam to fail, Petrolia Dam is considered unsafe until recommended actions (Section 3.2) are completed.

3.1.2 Spillway

The spillway system was designed to accommodate a relatively small flood event compared to current standards. Maximum spillway capacity, assuming the reservoir pool is at the first overtopping dam crest elevation, is approximately 10,200 cfs. The floodwater storage capability between the spillway crest and the first overtopping dam crest elevation amounts to 4608 AF. In comparison, the PMF for the 557 square-mile drainage area (which excludes the drainage area above Yellow Water Reservoir) is estimated to have a total runoff volume of about 616,700 AF. Hence, the combination of reservoir storage and spillway discharge capabilities is seriously inadequate to prevent overtopping of the dam during the PMF.

The spillway is in relatively poor condition with respect to concrete deterioration and the absence of support material under the spillway floor. The flowline of the spillway approach section is very irregular due to alternating degradation and aggradation processes. The approach channel banks have eroded to nearly a vertical orientation, particularly near the entrance to the concrete chute. Generally, the approach channel banks are not high and consequently a bank failure will not cause a major obstruction or threaten the dam embankment structure. Occasional maintenance is recommended, however. Also, undercutting and bank erosion have occurred in the stilling basin. The undercutting activity is not progressing at a critical rate, and the concrete chute and flip bucket are not endangered solely by this scour activity. However, remedial measures are recommended.

Bank erosion in the stilling basin is also progressing at a rate and in such a manner as not to effect structural stability and dam safety, but monitoring and stabilization measures are recommended.

3.1.3 Outlet Works

The outlet works facility appeared to be in satisfactory operating condition from the control tower to the outlet. The approach channel, inlet structure, 60-inch conduit from the inlet to the control tower, and the 30-inch outlet system to the high-line canal were not inspected. Minor repair is required on all of the gates, on the metal seat ring and concrete conduit downstream of the emergency and operating gates in the control tower, in the second wet-well chamber, at isolated sections of the 60-inch conduit below the second wet-well chamber, and in the outlet stilling basin. Rather severe undercutting and scouring have occurred below the outlet structure, and the rock riprap banks around the perimeter of the stilling basin have experienced minor subsidence. Minor repairs in the form of concrete rehabilitation and channel cleaning/regrading are needed at the entrance to the low-line canal.

3.1.4 Dam

Except for the loss of some riprap on the upstream face, the Petrolia Dam embankment appears to be stable and in good condition. Some questions exist as to the embankment stability because insufficient information is presently available to fully evaluate the dam. Settlement does not appear to be a problem. Numerous seepage areas were identified. Most seepage appears to be passing through the abutments or under the dam. The phreatic surface through the dam is unknown. The upstream face of the embankment is deficient in riprap protection in some areas as the riprap has moved down the slope exposing erodible soils. The dam crest width is less than shown on the construction drawing (Sheet 1 of Exhibit D1), and there are some depressions along the crest which collect water and provide a concentrated source of water for spillage over the dam face. As a result of this spillage, there are erosion gullies on the downstream face. A major erosion gully is located on the downstream side near the left (north) abutment contact.

3.1.5 Geology

Petrolia Reservoir basin is formed in a sandy shale sequence with 5 to 10-foot sandstone beds forming ledges throughout the area. The valley section consists of alluvial deposits of sandy gravel. Minor slope instability was noted along

the reservoir shoreline, but there was no evidence of large areas of instability in the basin. Petrolia Dam is located in a relatively quiet seismic zone according to a known distribution of damaging earthquakes and their associated intensities.

The only change in the site geology since the original investigations and reporting is the grouting which was performed in 1952 and 1953, and possibly again in the middle to late 1950's (available information is not conclusive on the second grouting program).

3.1.6 Operation and Maintenance

There are no formal operation plans and/or maintenance programs. The dam is visited regularly throughout most of the year by the ditch rider and/or water commissioner to regulate water releases. Regular project inspections are performed jointly by the DNRC and representatives of the local water users' association. Some repair work has been accomplished in the past, but not all project components have been maintained or repaired when first required. The basic reason for the meager maintenance program and repair efforts is the ever-present manpower and budgetary limitation. A warning system to alert downstream inhabitants of dam distress has not been developed.

3.2 Recommendations

(1) Immediately develop, implement, and periodically test an emergency warning plan for use in the event of dam distress. Combine this plan with the one developed for the Yellow Water Storage Project.

(2) Perform studies to determine the reasons for the spillway subgrade deterioration. Modify the spillway area as studies indicate to prevent future deterioration. Perform the followng repairs on the spillway: replace and regrade the gravel and cobble material where necessary in the approach channel; fill the scour hole immediately upstream of the ogee crest; repair concrete along the floor and walls, and at the flip bucket; repair the concrete on the spillway walls near the anchor bolts for the footbridge; provide relief for the standing water in the flip bucket; and repair the stilling basin. Stilling basin repairs are detailed in Recommendation #4. (See Appendix F for work done subsequent to the September 1979 inspection).

(3) Perform the following repairs on the outlet works facility: apply corrosion resistant protective coatings to all gates; repair the gate seat on the emergency slidegate; repair cavitation/erosion areas on the metal seat rings for the emergency and operating gates in control tower; repair the small leaks and erosion points in the concrete conduit; repair the concrete deterioriation in the second wet-well chamber (chamber housing the vertical turbine pump); and repair the stilling basin. Stilling basin repairs are detailed in Recommendation #4. (See Appendix F for work done subsequent to the September 1979 inspection).

(4) Repair and replace riprap in the following areas: along the upstream face where the embankment has been eroded and the rock has moved downslope; in the outlet works stilling basin where severe undercutting and erosion have occurred in the basin and where the rock sidewalls have subsided; and in the spillway stilling basin where channel degradation and localized bank erosion have occurred.

(5) Using suitable material, fill and properly compact all depressions along the dam crest.

(6) Install additional piezometers in the embankment, abutments, and foundation to evaluate the seepage conditions. Establish a regular monitoring program for the underseepage and abutment seepage. Particular attention should be given to periods when the reservoir pool is relatively high. The results of the monitoring program will be used in Recommendation #9.

(7) Repair all erosion gullies on the downstream embankment face, particularly the major gully along the downstream side of the left abutment contact.

The above recommendations will not make the project safe but will reduce involved risks while the following recommendations with subsequent actions are being accomplished.

(8) Conduct more detailed hydrologic and hydraulic routing studies to better determine the downstream hazard and required spillway capacity. Include input from the upstream Yellow Water Reservoir in these studies and modify the project as studies indicate.

(9) Conduct and place on file, stability and seepage analyses of the embankment. It is recommended that these analyses be performed by a qualified geotechnical engineer and be based on: static loading conditions;

in situ strength properties of the embankment, foundation, and abutment materials; and phreatic surface conditions. Establish the material strength properties by drilling and sampling with laboratory testing as appropriate; and obtain the phreatic surface by evaluating the results of the piezometer monitoring program. Modify the project as studies indicate.

(10) Monitor the bank erosion and sloughing on the left stilling basin wall of the spillway and perform the necessary rehabilitation.

(11) Determine whether the cutoff wall on the spillway crest contacts bedrock, and assess the possibility of flow passing under the cutoff.

(12) Conduct periodic inspections by qualified engineers at least once every five years to determine whether there are any deficiencies in the condition of the project, to assess the adequacy and quality of maintenance, and to evaluate methods of operation. Include an inspection of all conduits through the embankment.

(13) Develop and implement a periodic maintenance plan for the dam and appurtenant structures.

Prior to performing engineering studies and remedial construction, the project operator should coordinate the work with the Montana DNRC, Dam Safety Section, to insure compliance with all pertinent laws and regulations.

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2. U.S. Geological Survey, 7½-Minute Quad Maps, Petrolia Lake, Montana, 1963, Breed Spring, Montana, 1963, Mosby, Montana, 1962.

3. Montana Water Resources Board, Montana Register of Dams, Inventory Series No. 3, October, 1968.

4. Montana Department of Natural Resources and Conservation, Water Resources Division, Progress Report: Project Critiques, Supplement "A", January 1973.

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19. U.S. Geological Survey, _Water Resources Data for Montana, - 1977_.

20. National Weather Service, _Technical Paper No. 38, Generalized Estimates of Probable Maximum Precipitation for the United States West of the 105th Meridian for Areas to 400 Square Miles and Durations to 24 Hours_, Washington, D.C., 1960.

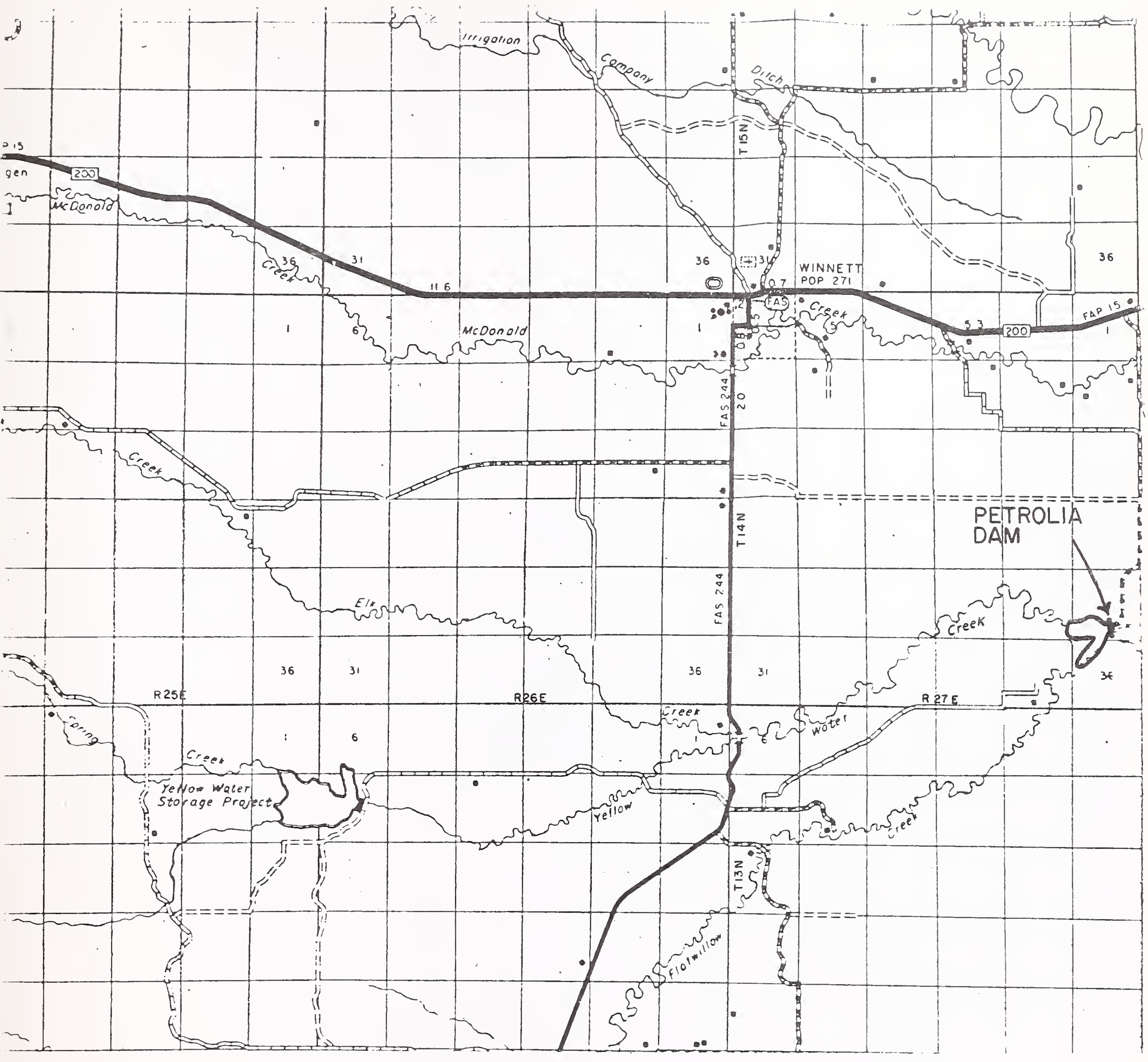
21. National Weather Service, _Hydrometerorological Report No. 43, Probable Maximum Precipitation, Northwest States_, Washington, D.C., 1966.

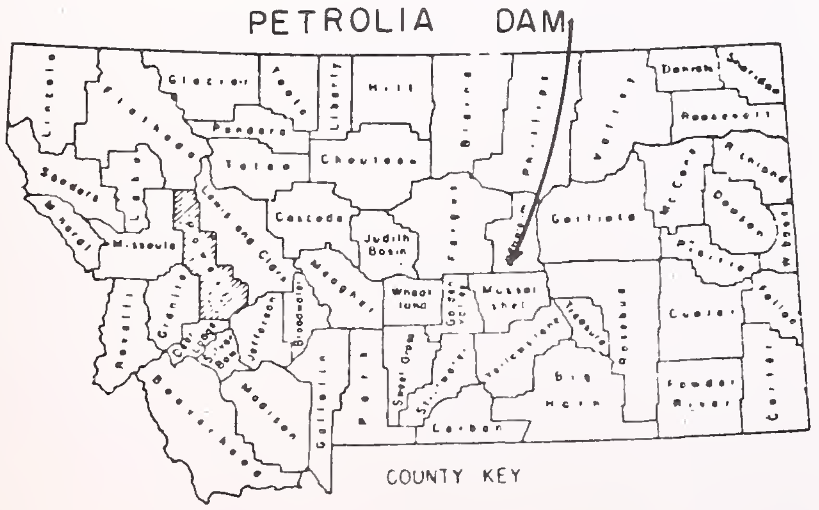
22. U.S. Soil Conservation Service, _National Engineering Handbook_, Section 4, "Hydrology", Washington, D.C., 1969.

23. U.S. Army Corps of Engineers, Hydrologic Engineering Center, _HEC-1 Flood Hydrograph Package_, Davis, California, September, 1978.

APPENDIX A

PETROLIA DAM
VICINITY MAP



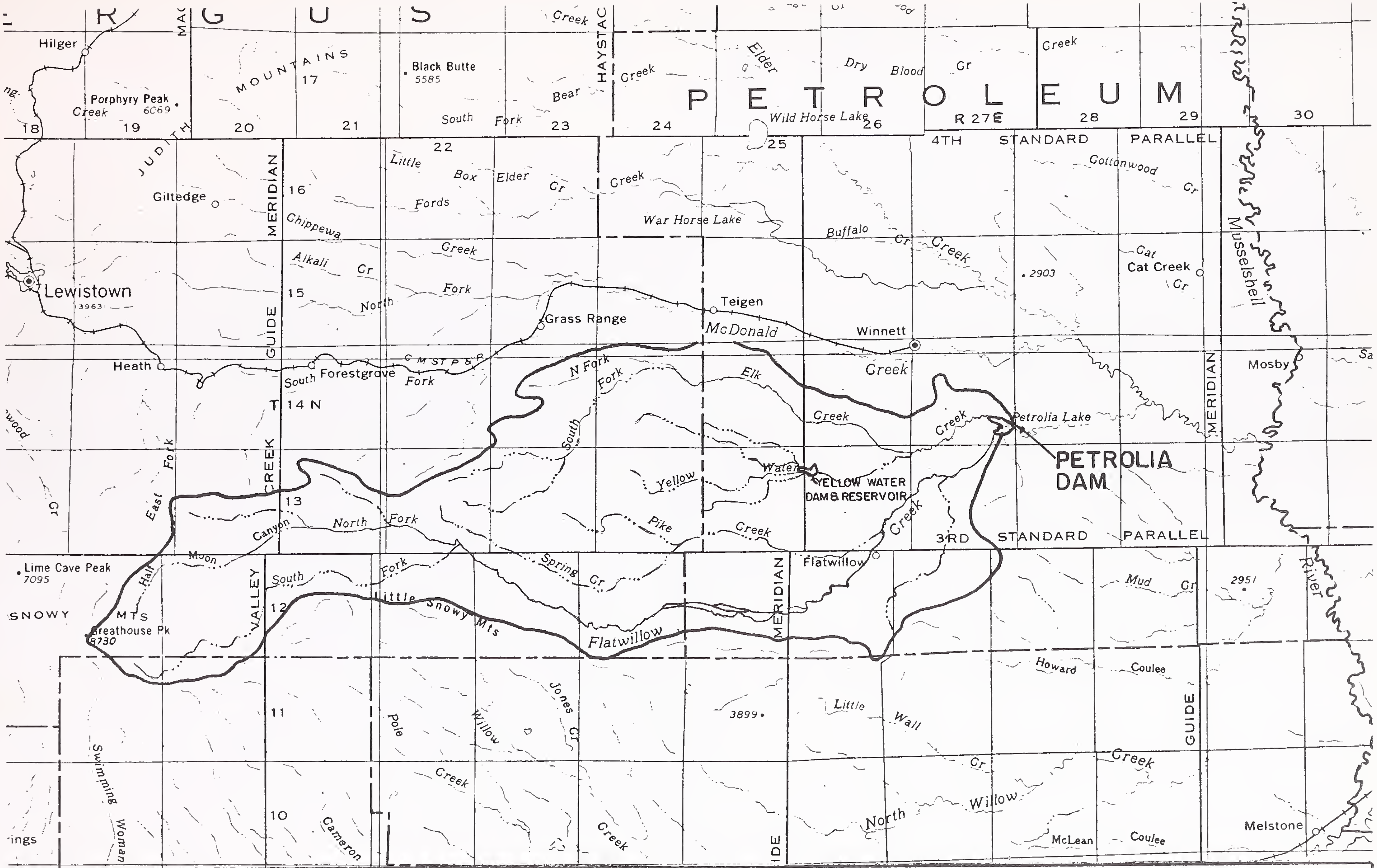


SOURCE: GENERAL HIGHWAY MAP, (1973)
PETROLEUM COUNTY, MONTANA
MONTANA STATE HIGHWAY COMMISSION

APPENDIX A
VICINITY MAP
PETROLIA DAM

APPENDIX B

PETROLIA DAM
WATERSHED MAP



SOURCE: STATE OF MONTANA
U.S.G.S. 1966

APPENDIX B
WATERSHED MAP
PETROLIA DAM

APPENDIX C

PETROLIA DAM
INSPECTION PHOTOS

Photo No. 1 - The Reservoir Basin
Standing on the dam crest looking westerly toward the upper end of the reservoir.

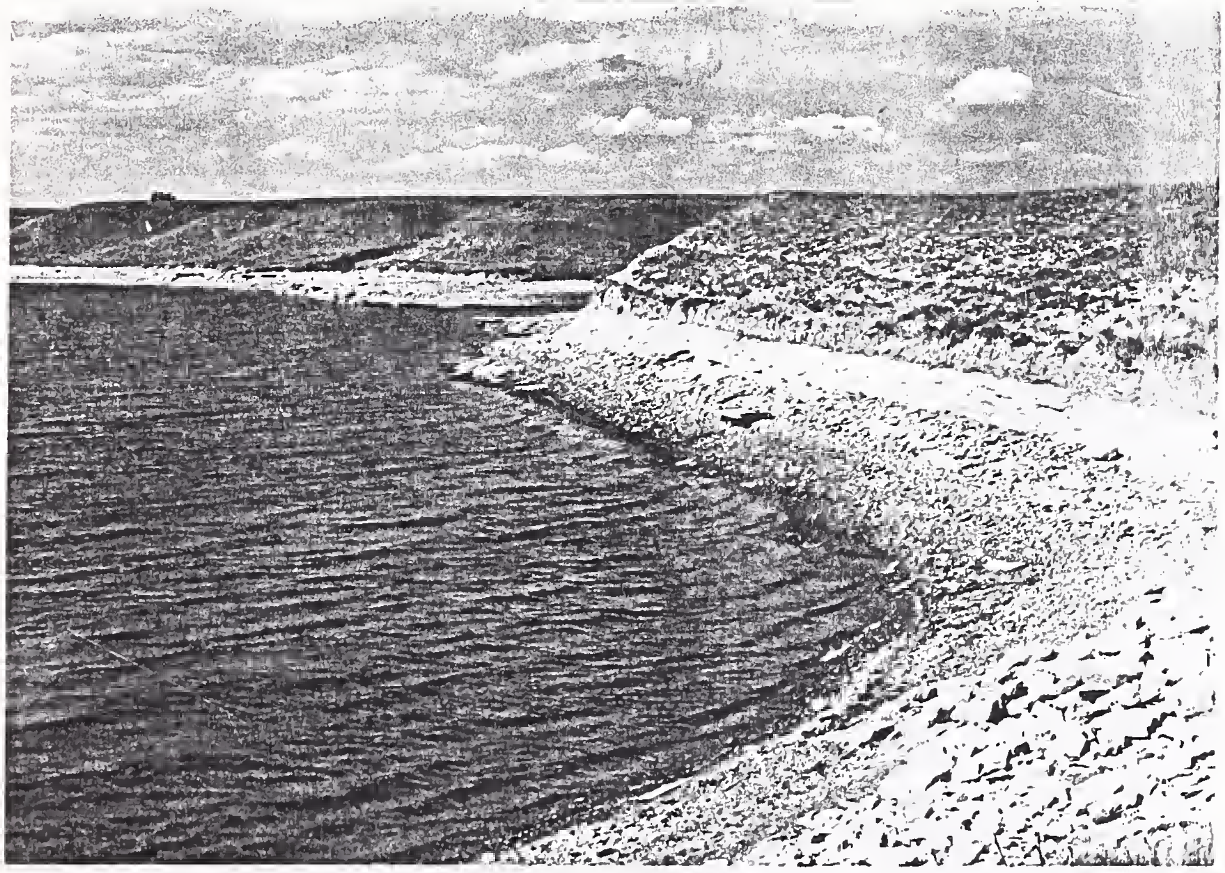


Photo No. 2 - The North Shoreline
Bedrock is exposed near the high water elevation. Overburden soils are standing near vertical above the bedrock.

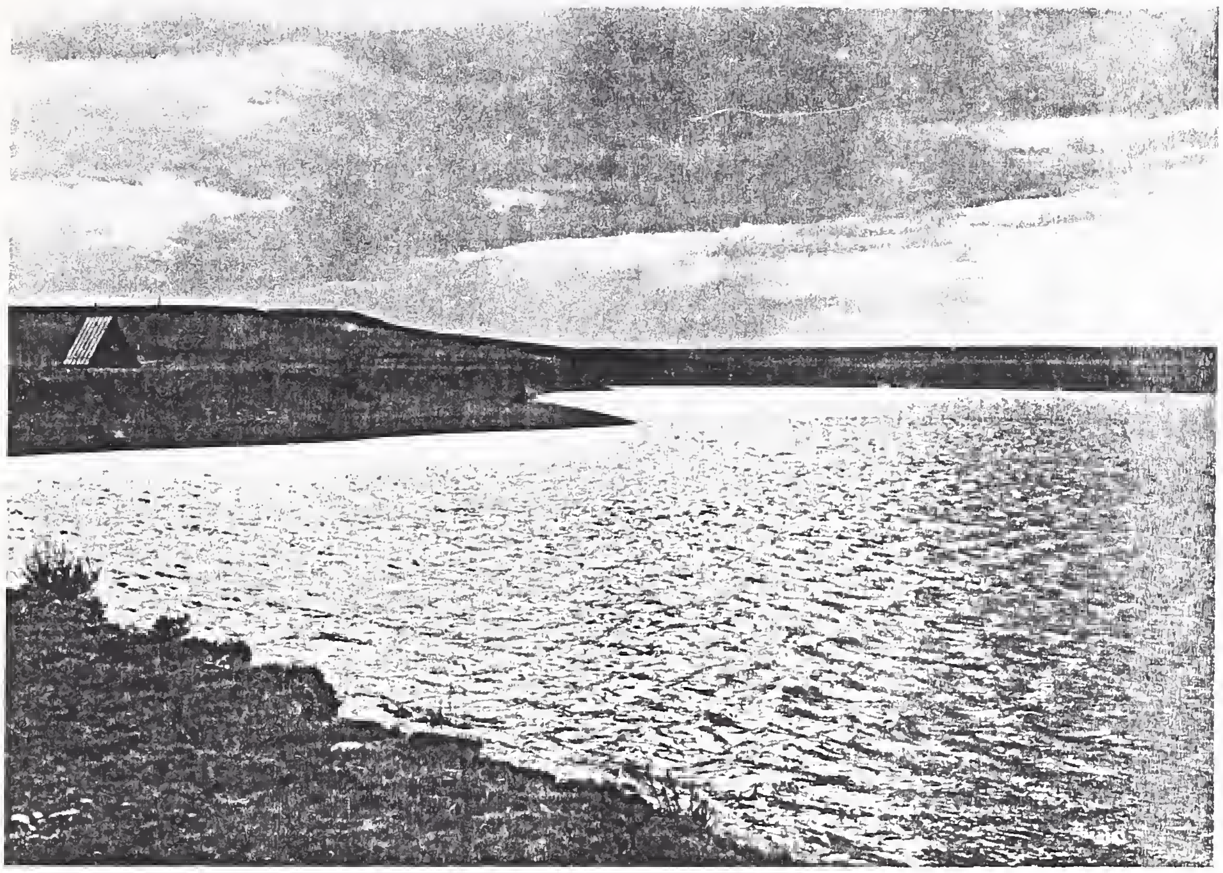


Photo No. 3 - The South Shoreline
High vertical banks are visible.

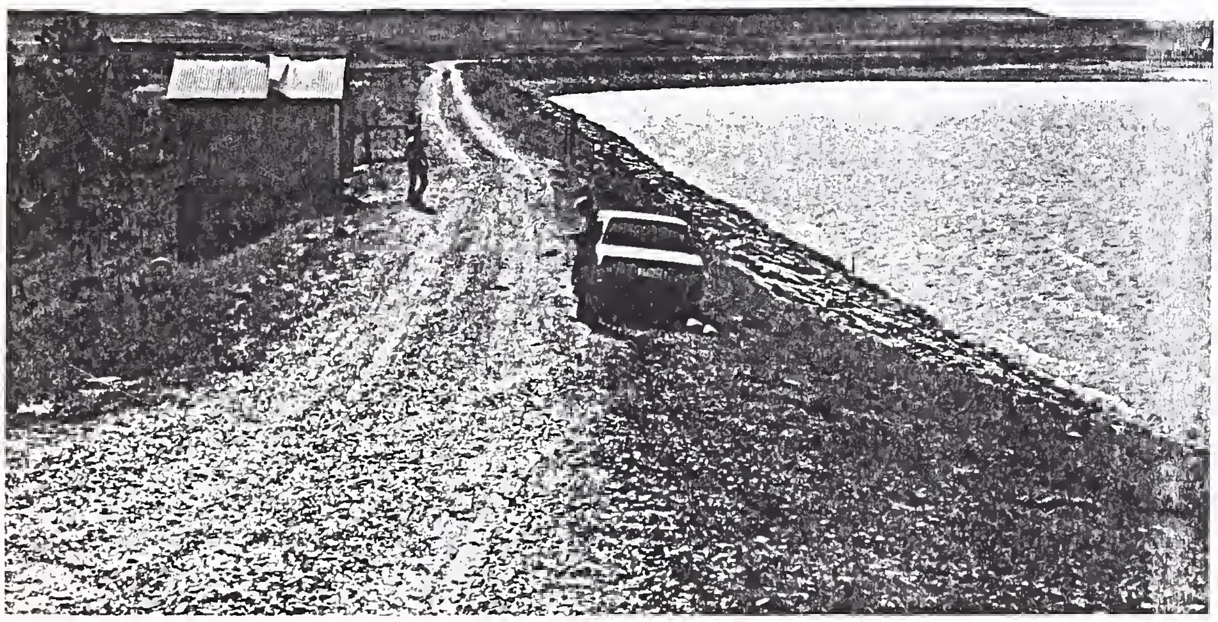


Photo. No. 4 - Dam Crest
Standing at the left abutment contact looking south along the crest. Notice the crest turns westerly near the center of the dam. Valve control house is in the left-center of photo.

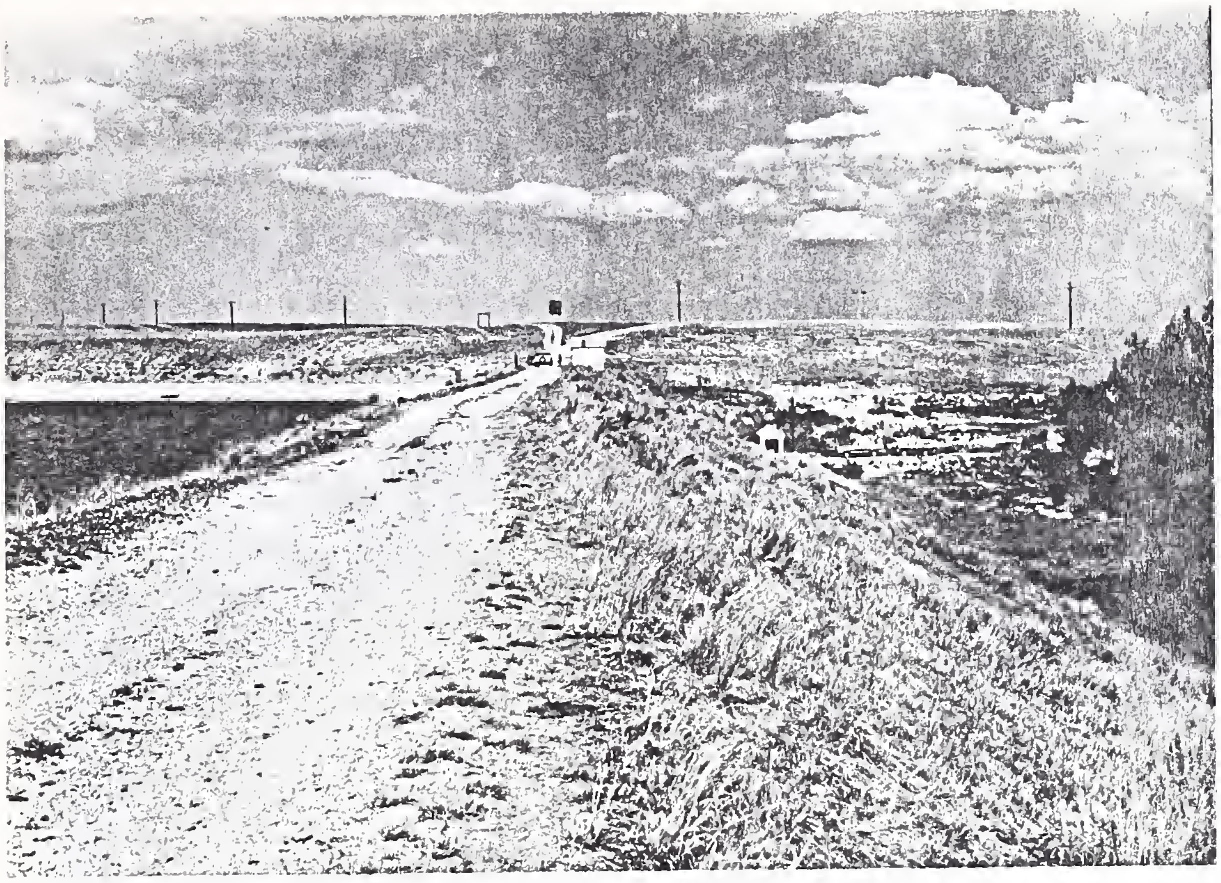


Photo No. 5 - Dam Crest.
Looking north toward the left abutment. The pump and control structure for the low-line irrigation canal is located near the downstream toe at the left abutment. Low-line canal is near vehicles; high-line canal is visible above structure and vehicles.

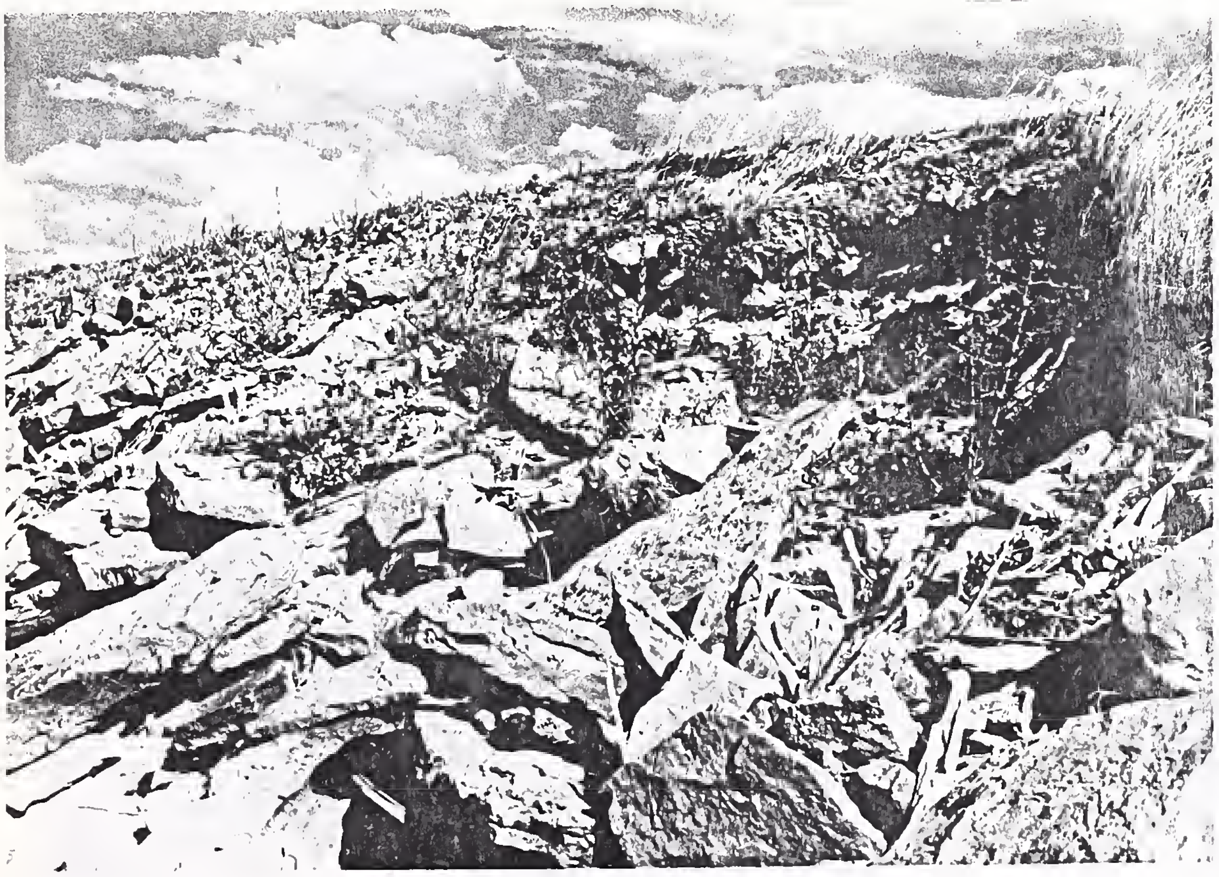


Photo No. 6 - Oversteepened Upstream Slope.
In some places the riprap on the upstream face has been undercut by runoff and the material has slid down leaving the slope exposed and oversteepened. This may also occur during high water. Notice small debris accumulations.

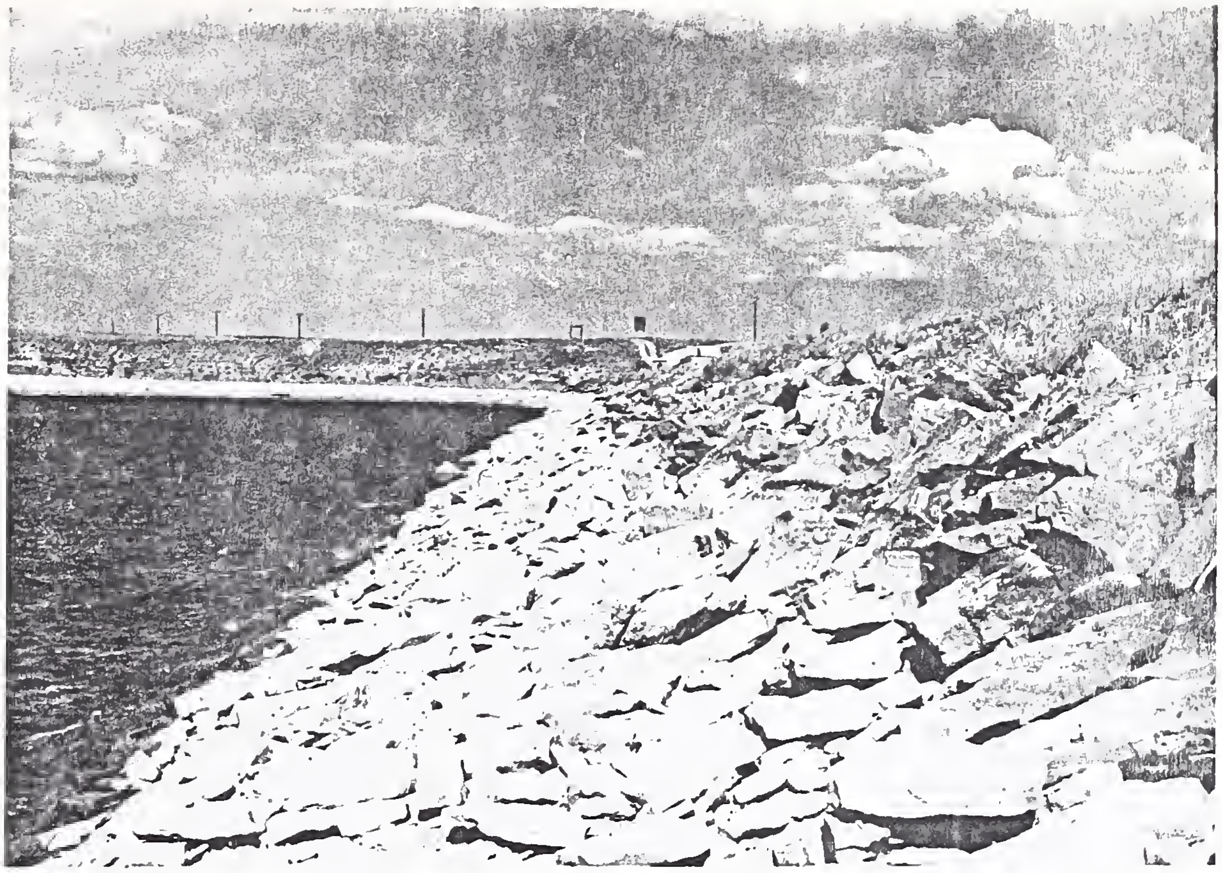


Photo No. 7 - Riprap on Upstream Face.
The riprap is predominantly a laminated sandstone. Normal high water mark is visible.

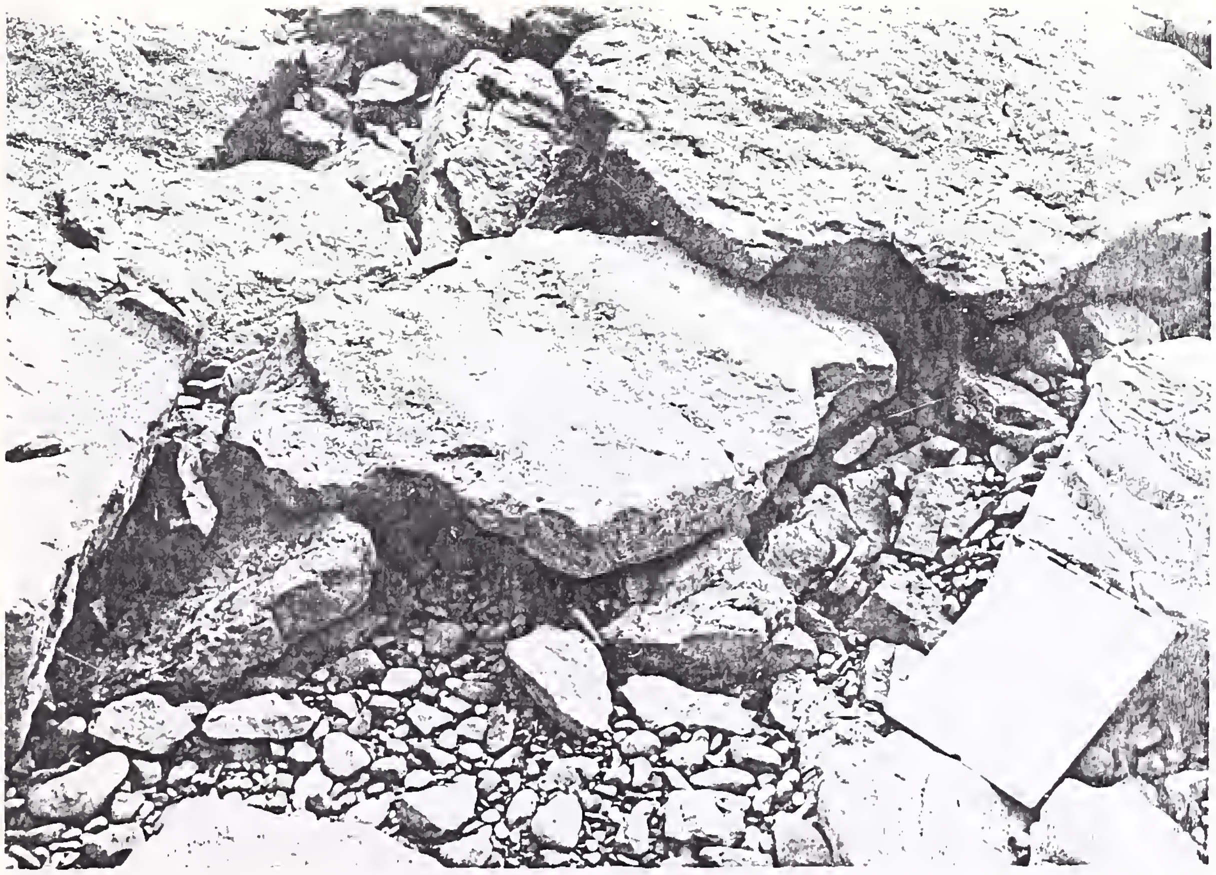


Photo No. 8 - Riprap Deterioration.
The riprap is a laminated sandstone with a mean size of about 12 inches. At the lower elevations, this material is weathered. The sandstone is becoming rounded and laminations are breaking off. Presently, the riprap is adequate.

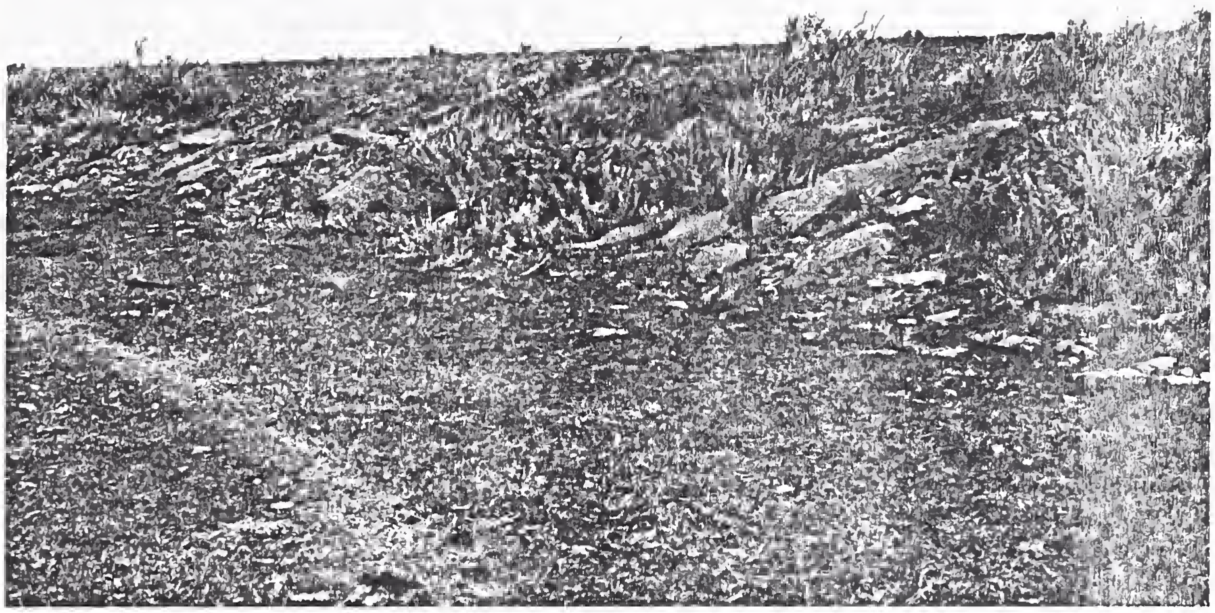


Photo No. 9 - Riprap on Upstream Face.
Picture taken of the south end of the upstream embankment face. Notice the sparcity of riprap. Considerable vegetation has grown around the riprap.

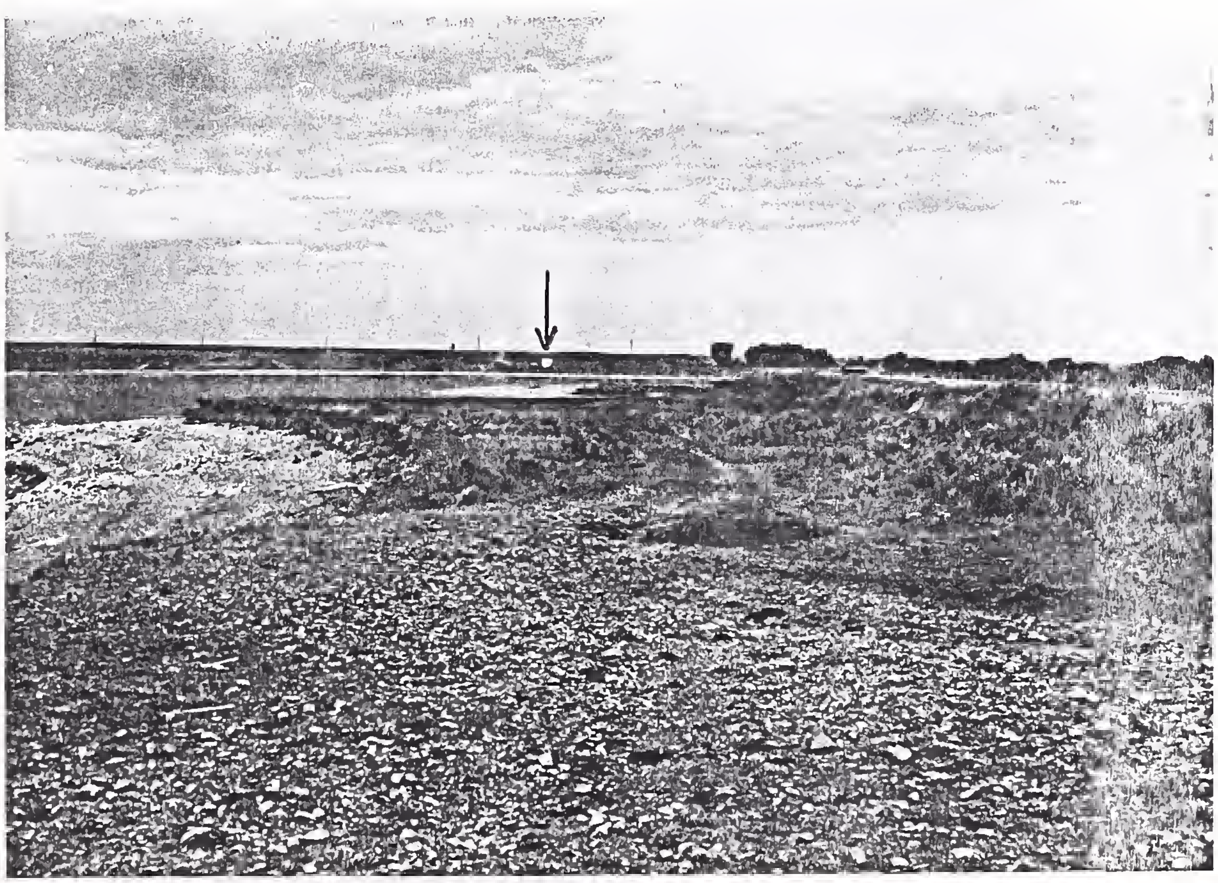


Photo No. 10 - The Upstream Slope.
Looking northeasterly. The upstream slope is composed of natural gravels, clay, and bedrock. Notice the structure housing the gate controls near the left abutment on the main embankment (see arrow).

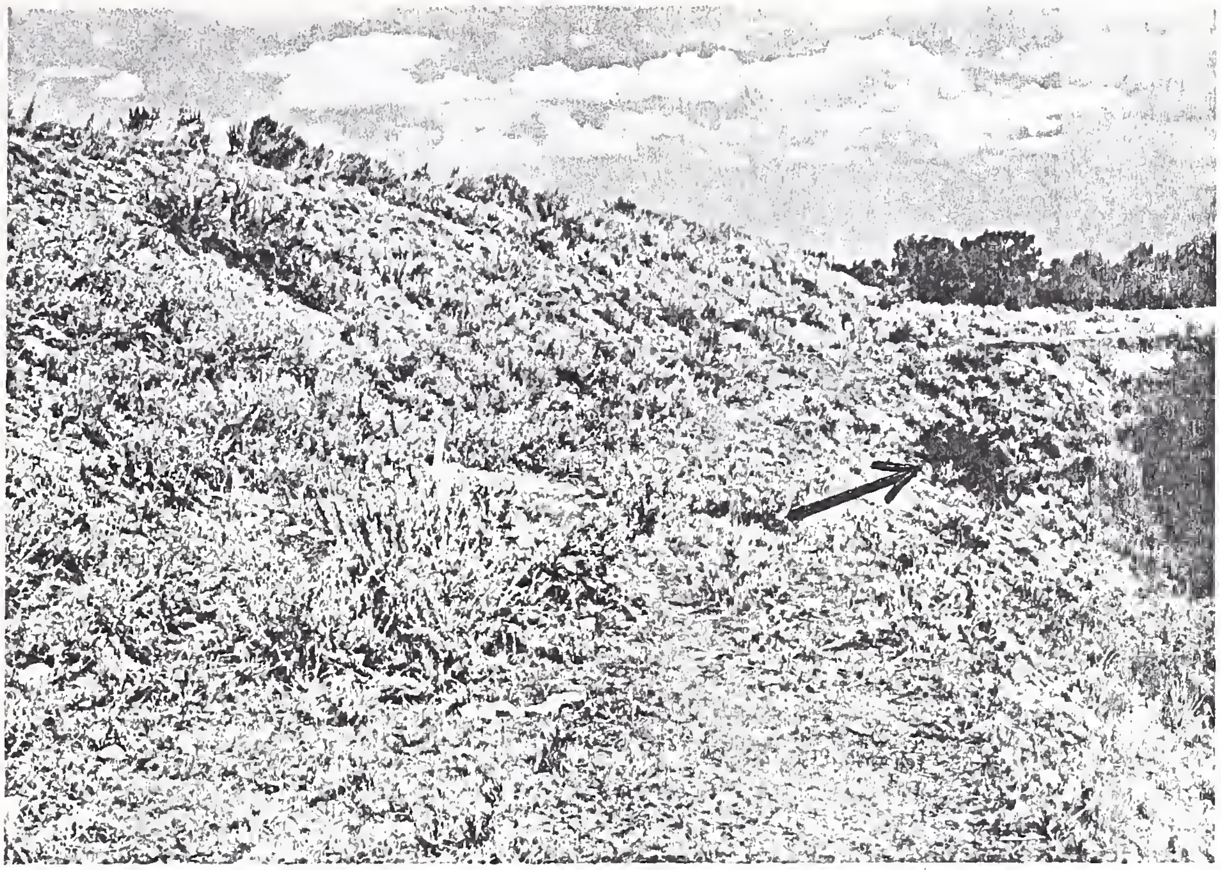


Photo No. 11 - Downstream Face and Groundwater Monitoring
The dam embankment is on the left in this photo with the toe of the fill near the white plastic groundwater monitoring pipe. The natural slope continues to the right. The bush identified with the arrow is also shown in Photo No. 13.

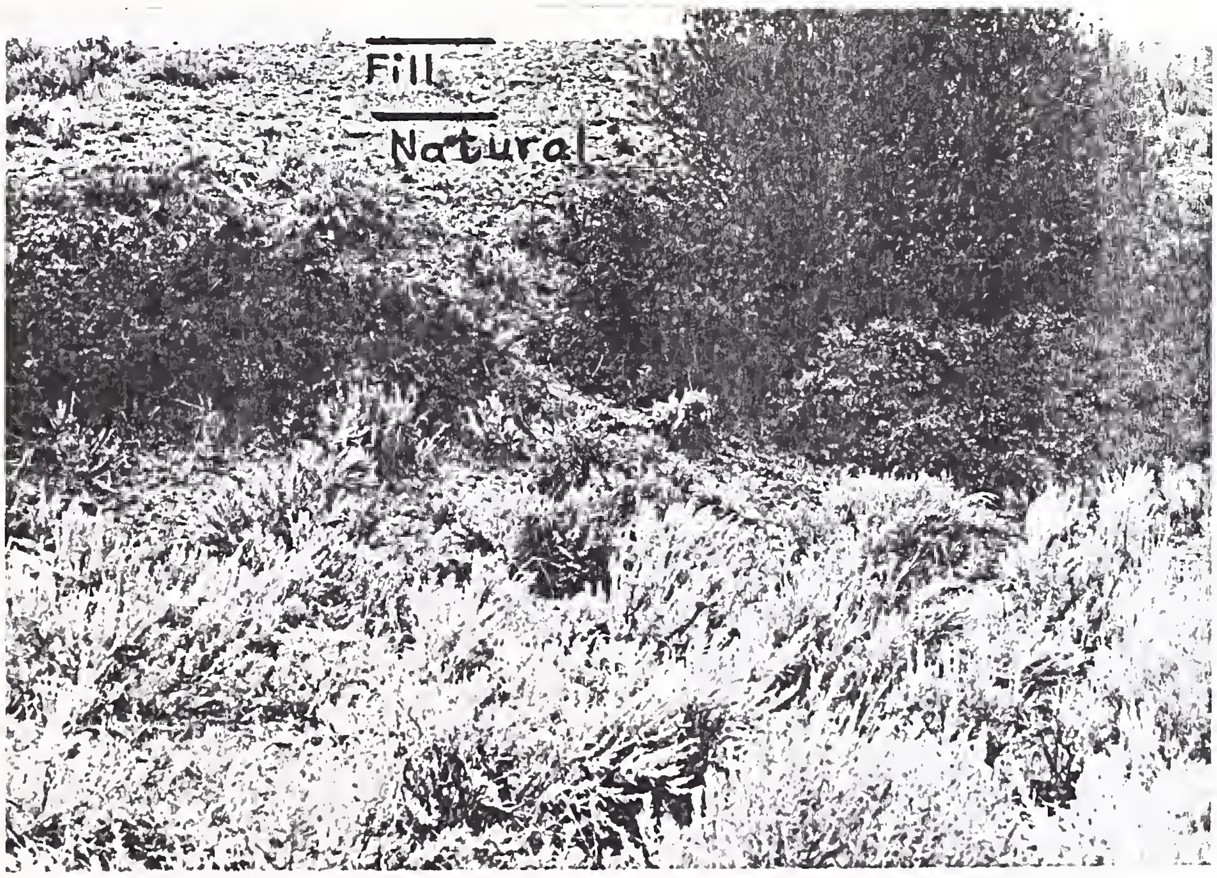


Photo No. 12 - High Groundwater Due to Seepage
First of a 3-picture series of the downstream embankment face. Notice the fill portion and the natural soil portion of the slope. Groundwater is high in this area as a result of seepage through the natural material.

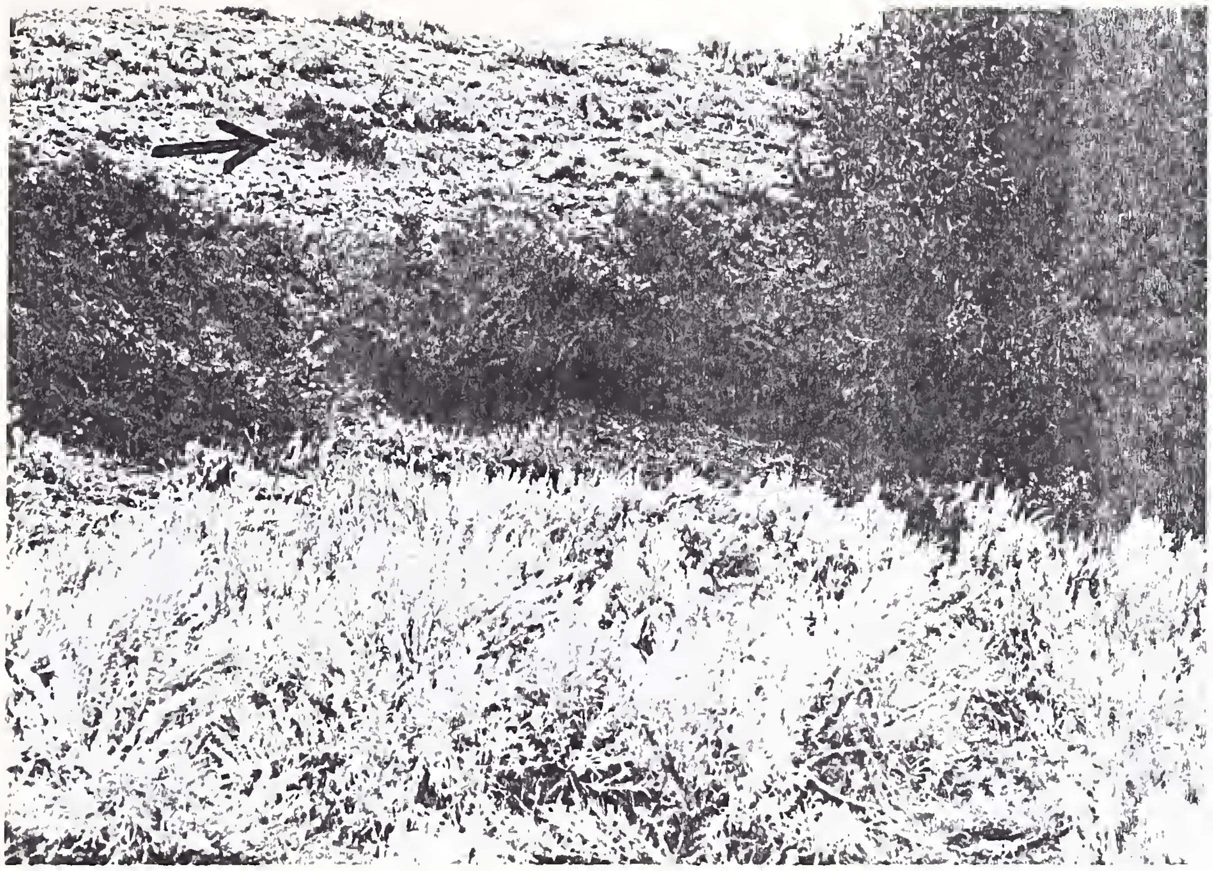


Photo No. 13 - High Groundwater Due to Seepage
Second in a 3-picture series. The bush identified with the arrow is also shown in Photo No. 11.

Photo No. 14 - High Groundwater Due to Seepage
Third in a 3-picture series.

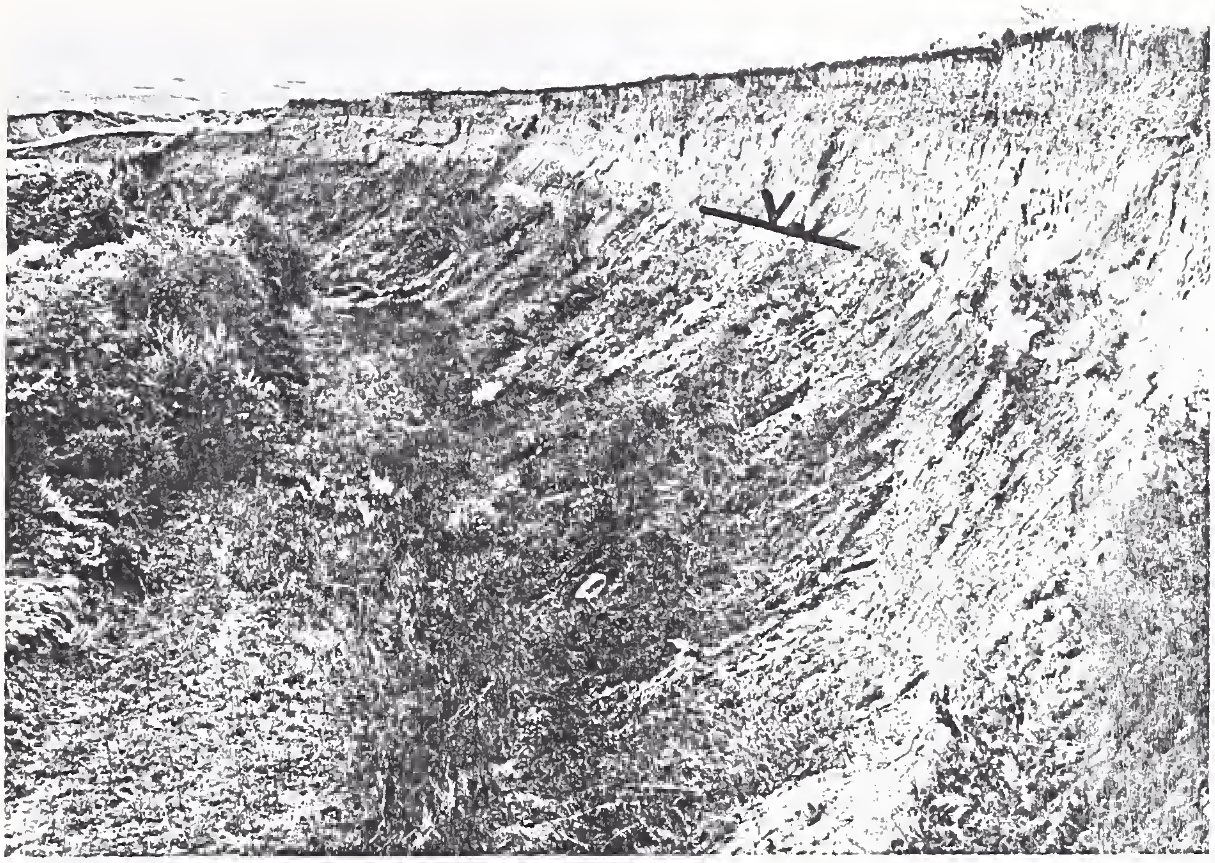


Photo No. 15 - Seepage
Seepage is exiting on the bank up to an elevation identified by the line. This bank is located about 100 to 200 feet east of the embankment and directly south of the area identified in Photo No.'s 11-14.

Photo No. 16 - Spring Activity.
A small boil is shown here located in the creek channel about 300 feet east of the embankment near the area identified in Photo No.'s 11-14.

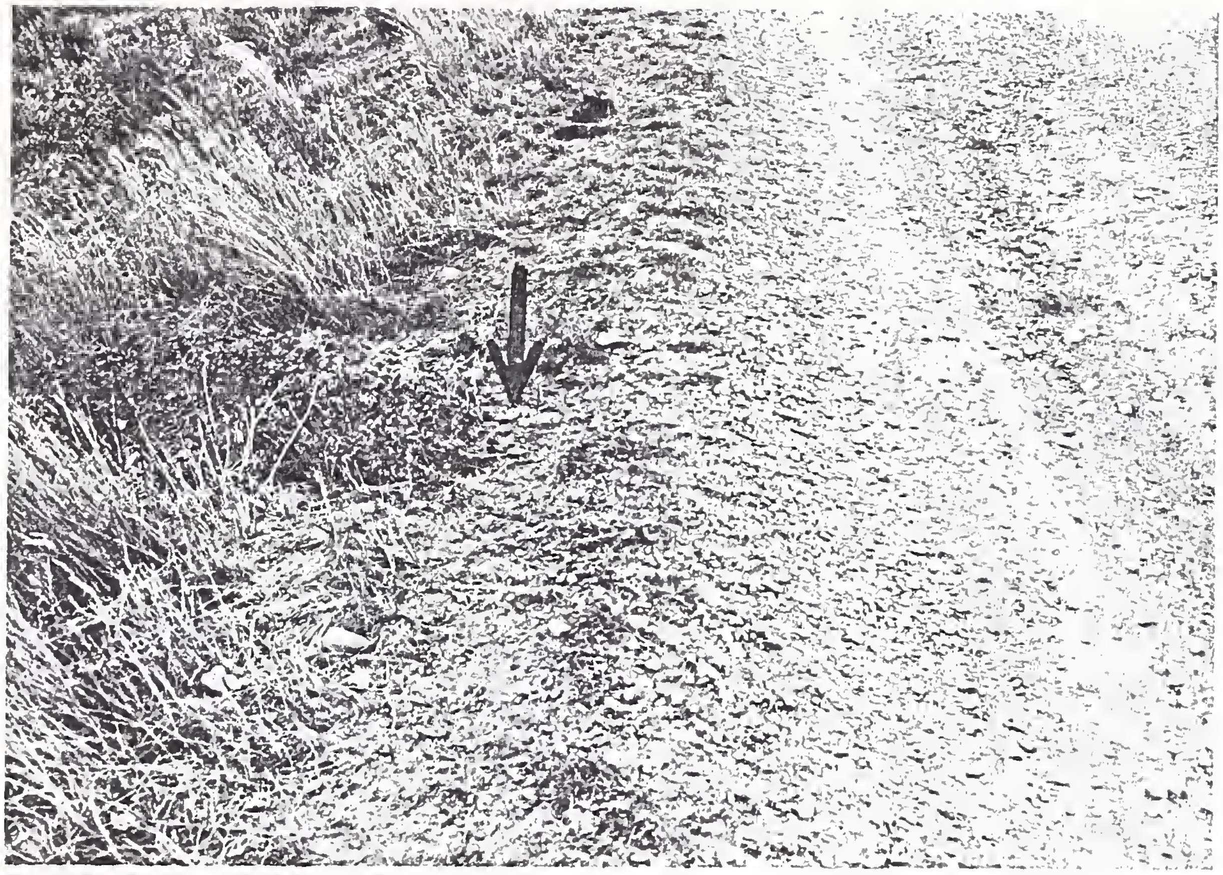


Photo No. 17 - Surface Erosion on Dam
Local water accumulations on the crest of the embankment ponds in the road (right) and runs downslope to the left (see arrow). Small erosion channels have been formed in these areas.

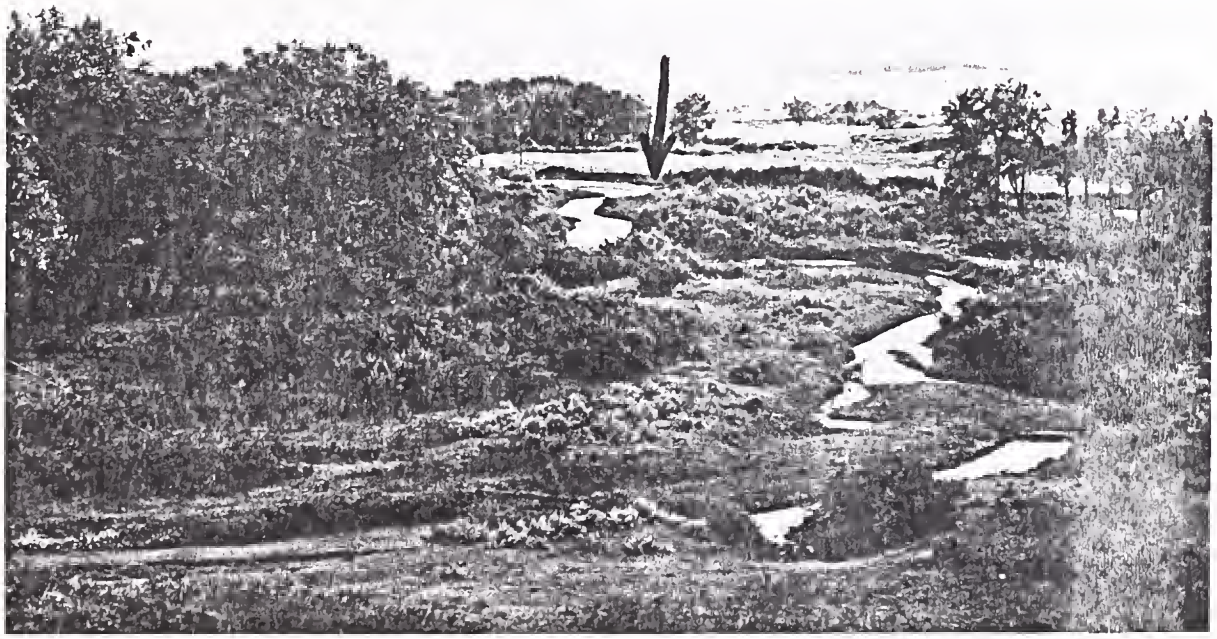


Photo No. 18 - Downstream Channel and Valley
The valley floor is wide and relatively deep. There are numerous trees and bushes on the valley floor. The confluence of the outlet channel (foreground) and the spillway channel is identified by the arrow.

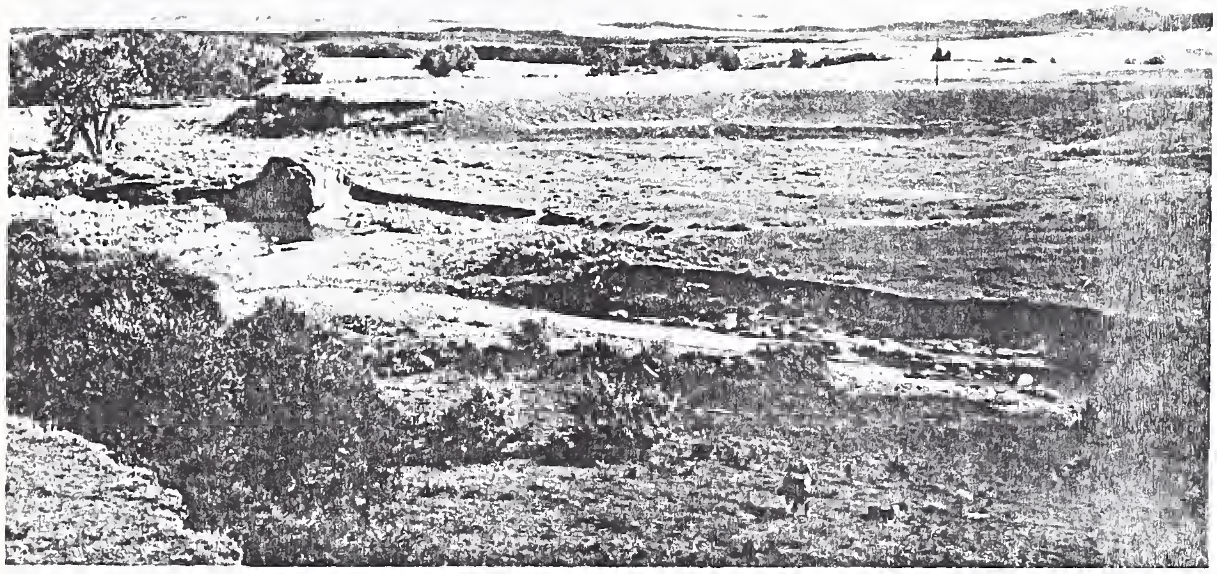


Photo No. 19 - Downstream Valley
Looking northeast from the embankment. The spillway return channel is shown in the foreground.

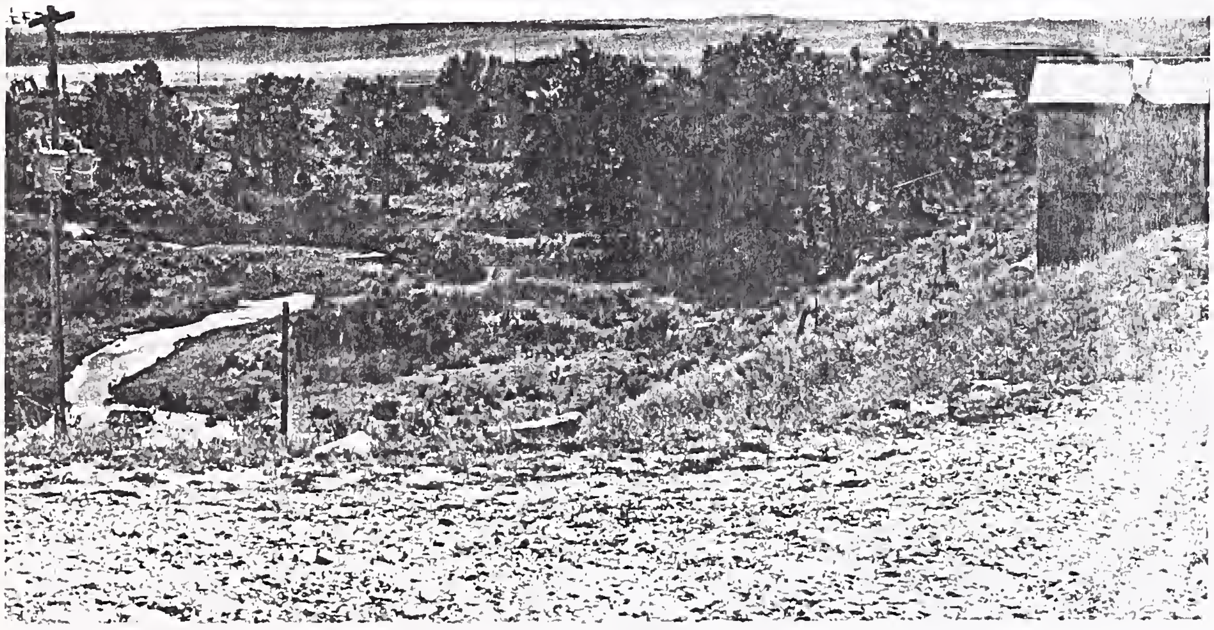


Photo No. 20 - Outlet Works Control House and Return Channel
Standing at the left abutment contact looking at part of the downstream face of the dam and the structure housing the gate controls. Return channel is shown in left-center of photo.

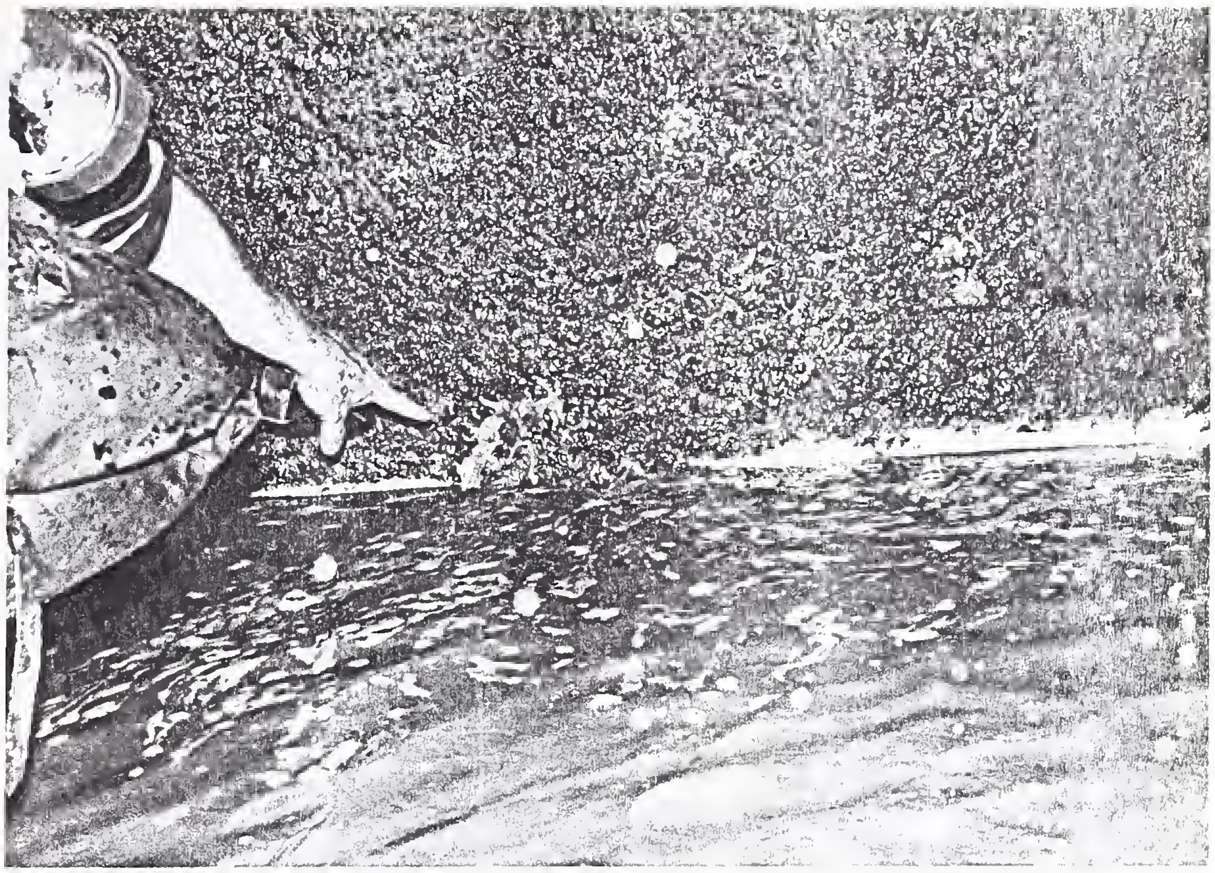


Photo No. 21 - Leak Under the Emergency Gate
There is a leak in the seal at the base of the emergency slide gate. Some corrosion of the gate is visible.

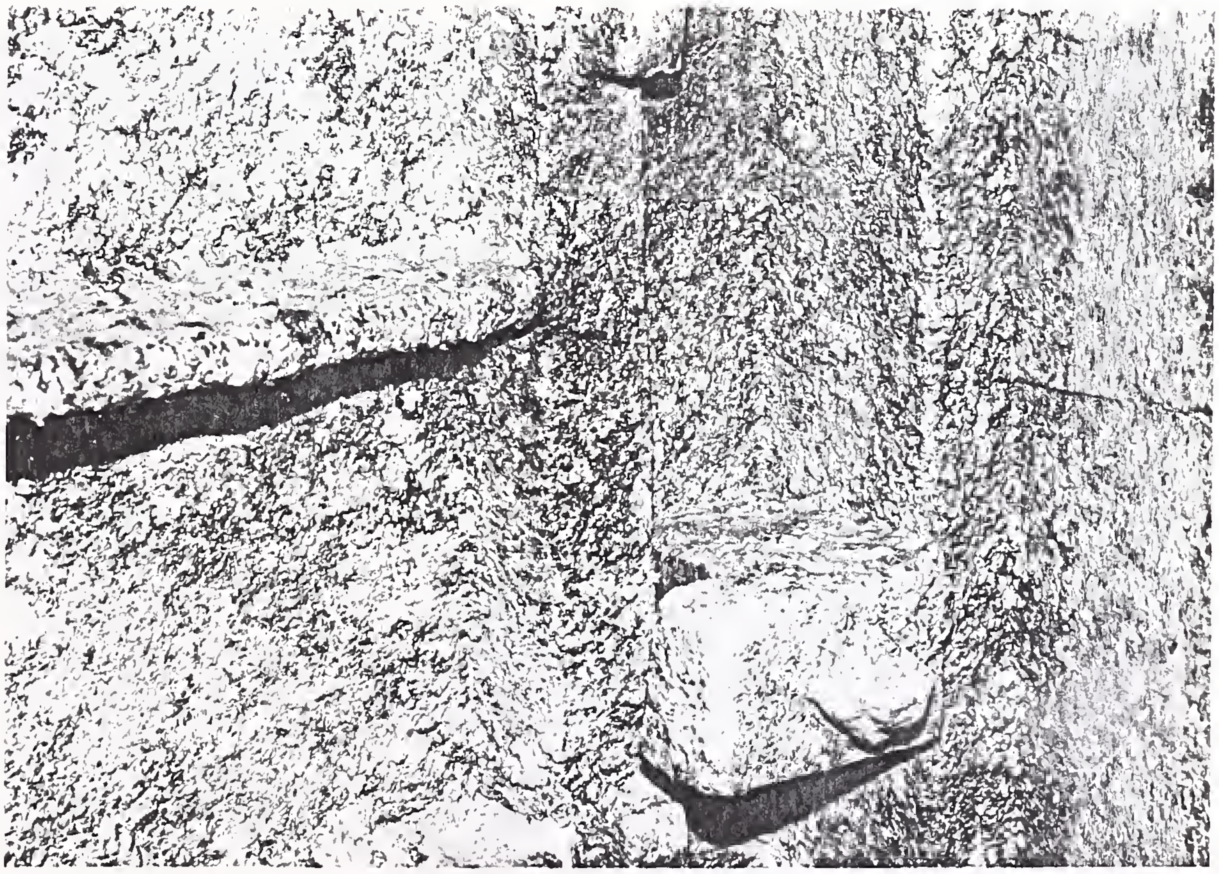


Photo No. 22 - Operating Gate and Guides
The operating gate has experienced moderate corrosion. A slide guide is visible.

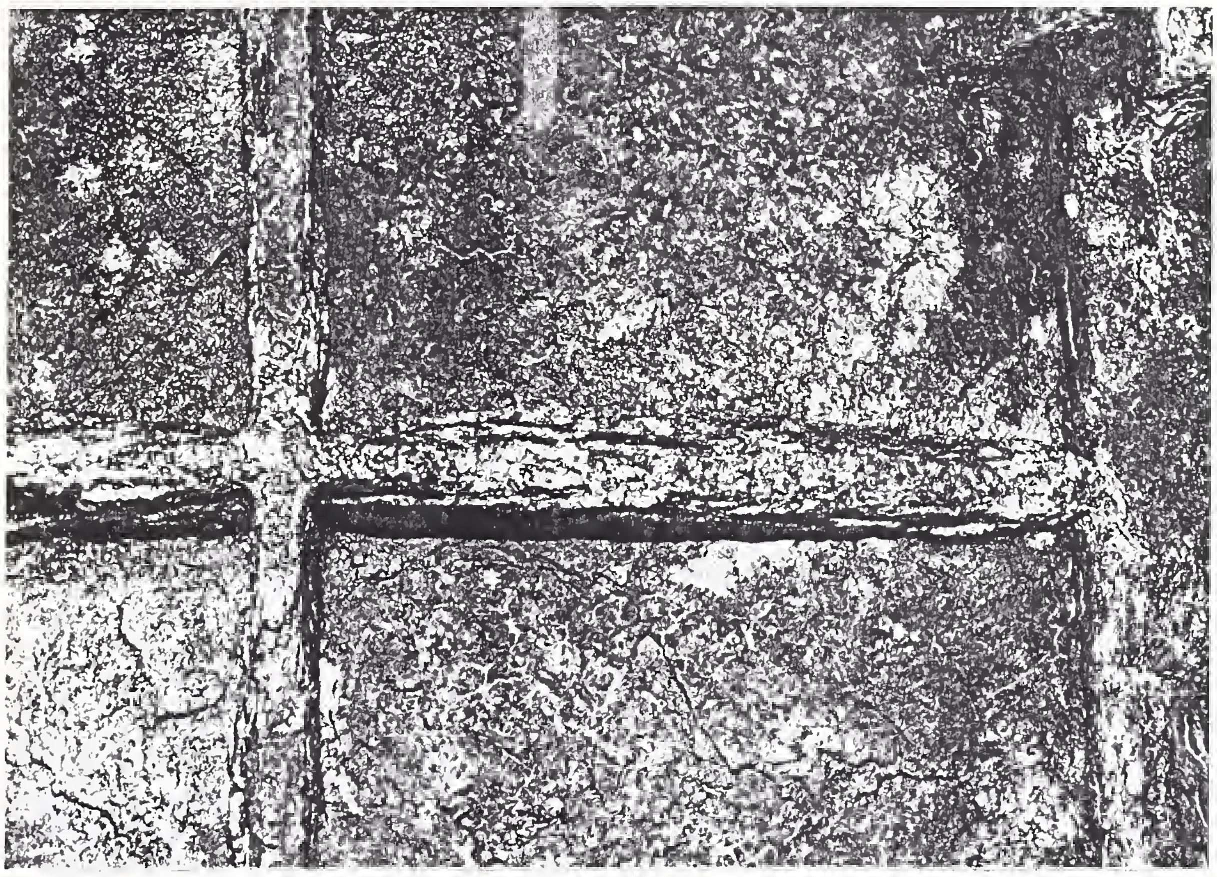


Photo No. 23 - Irrigation Sump Control Gate for Low-Line Canal
This gate has experienced a significant amount of corrosion.
The ribbing is weak and the metal is flaking.

Photo No. 24 - Operating Gate
This photo shows pitting of the iron pipe inside the conduit
just below the operating gate.

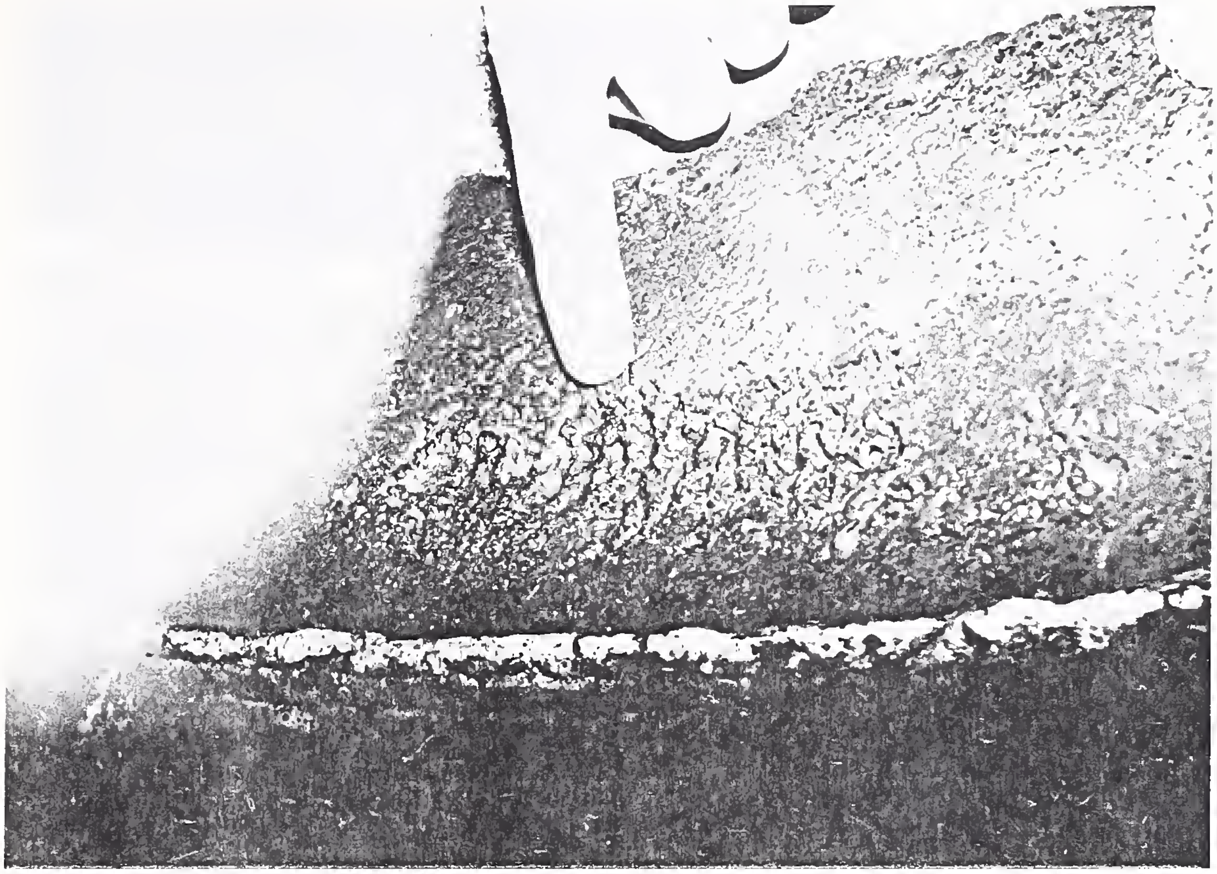


Photo No. 25 - Operating Gate
This photo shows pitting of the iron pipe inside the conduit just below the operating gate.

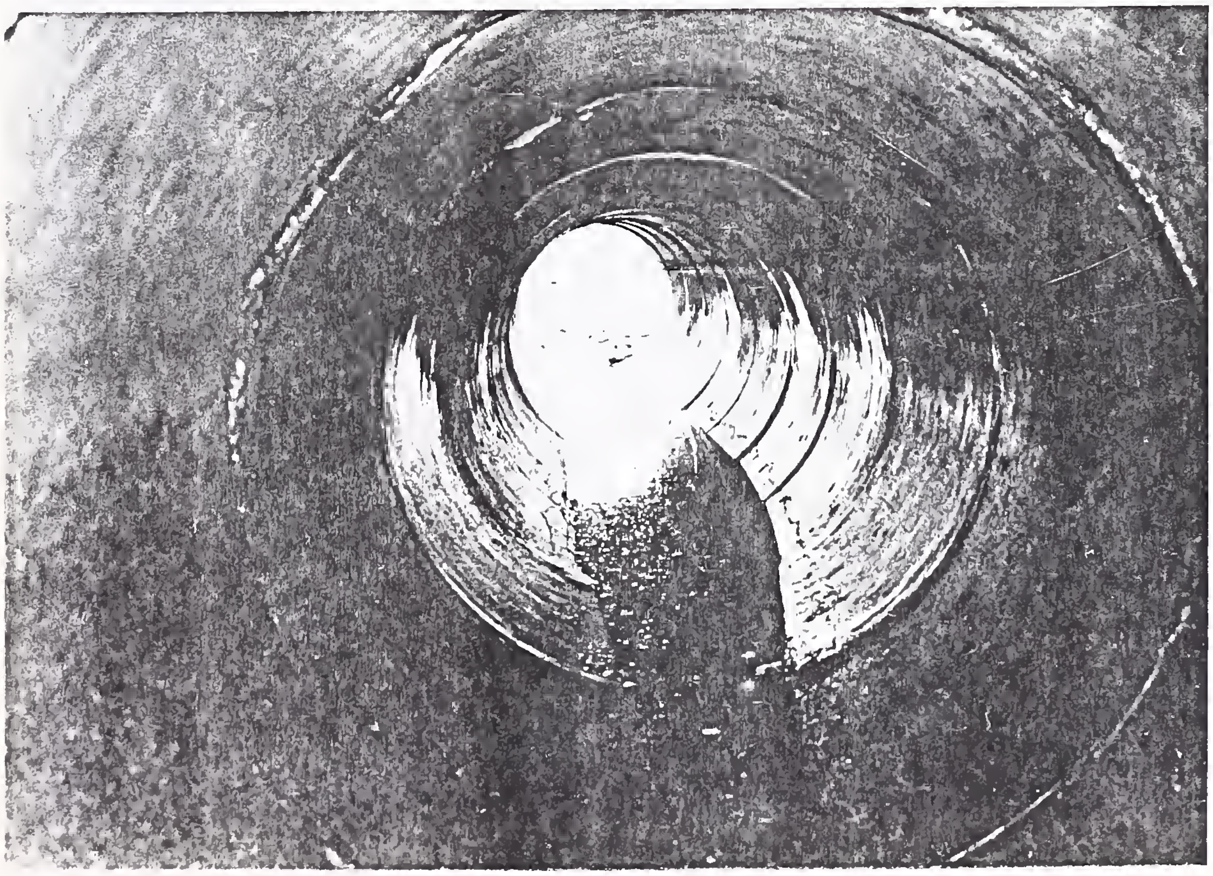


Photo No. 26 - The Outlet Conduit
The standing water in the conduit is the result of differential settlements or poor construction quality control. The conduit is concrete. The concrete joints have been filled and sealed.

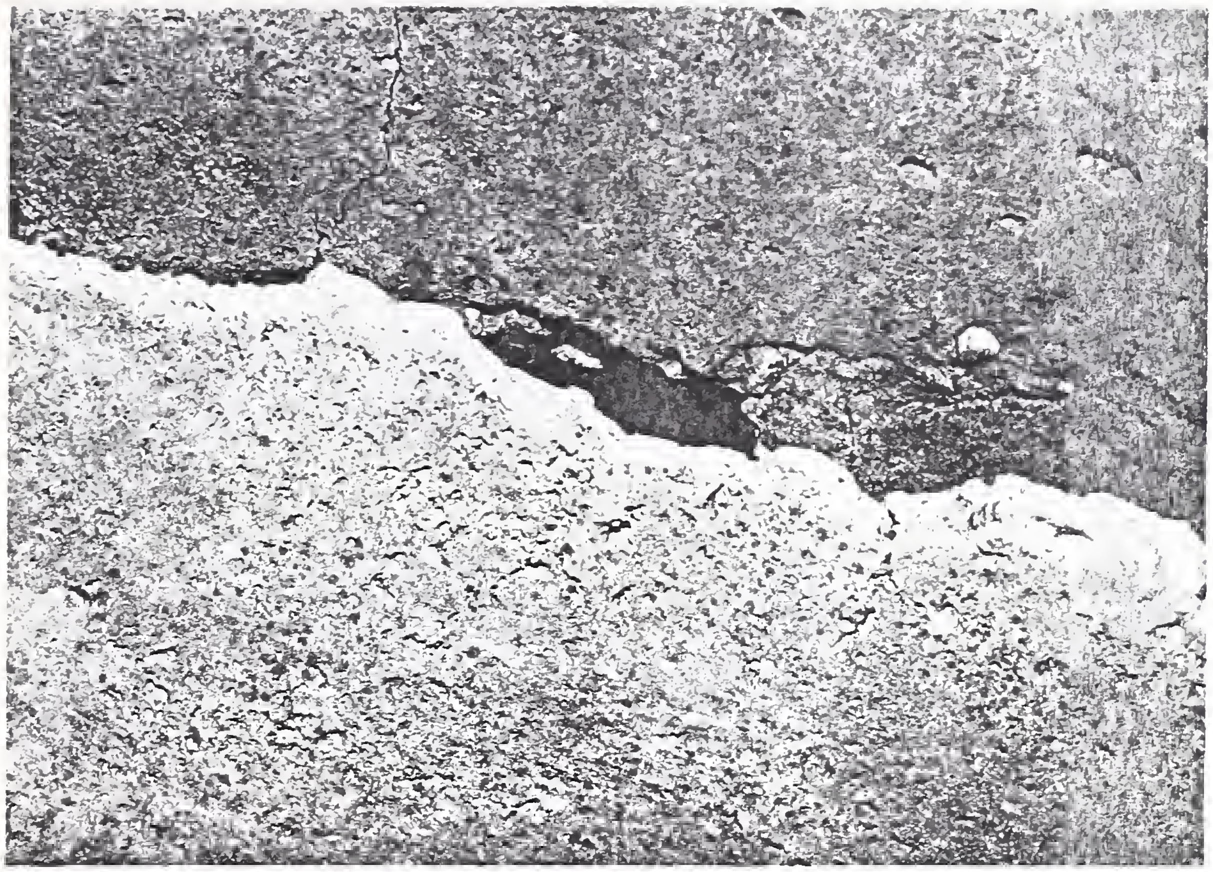


Photo No. 27 - Hole in the Irrigation Sump Structure Wall
The depth of the hole could not be determined. A piece of rebar is exposed in the hole.

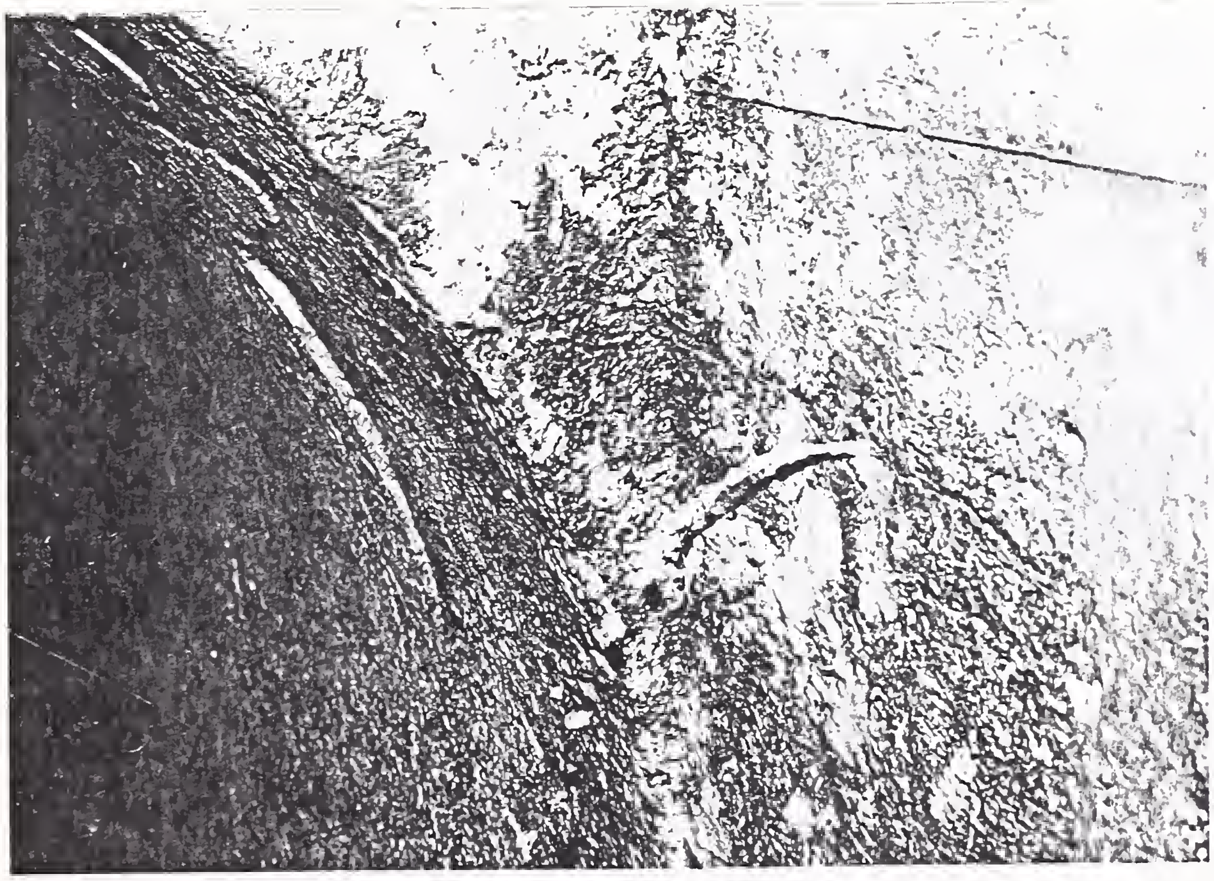


Photo No. 28 - Leak in the Irrigation Sump Structure Wall
A crack and leak is shown in the concrete

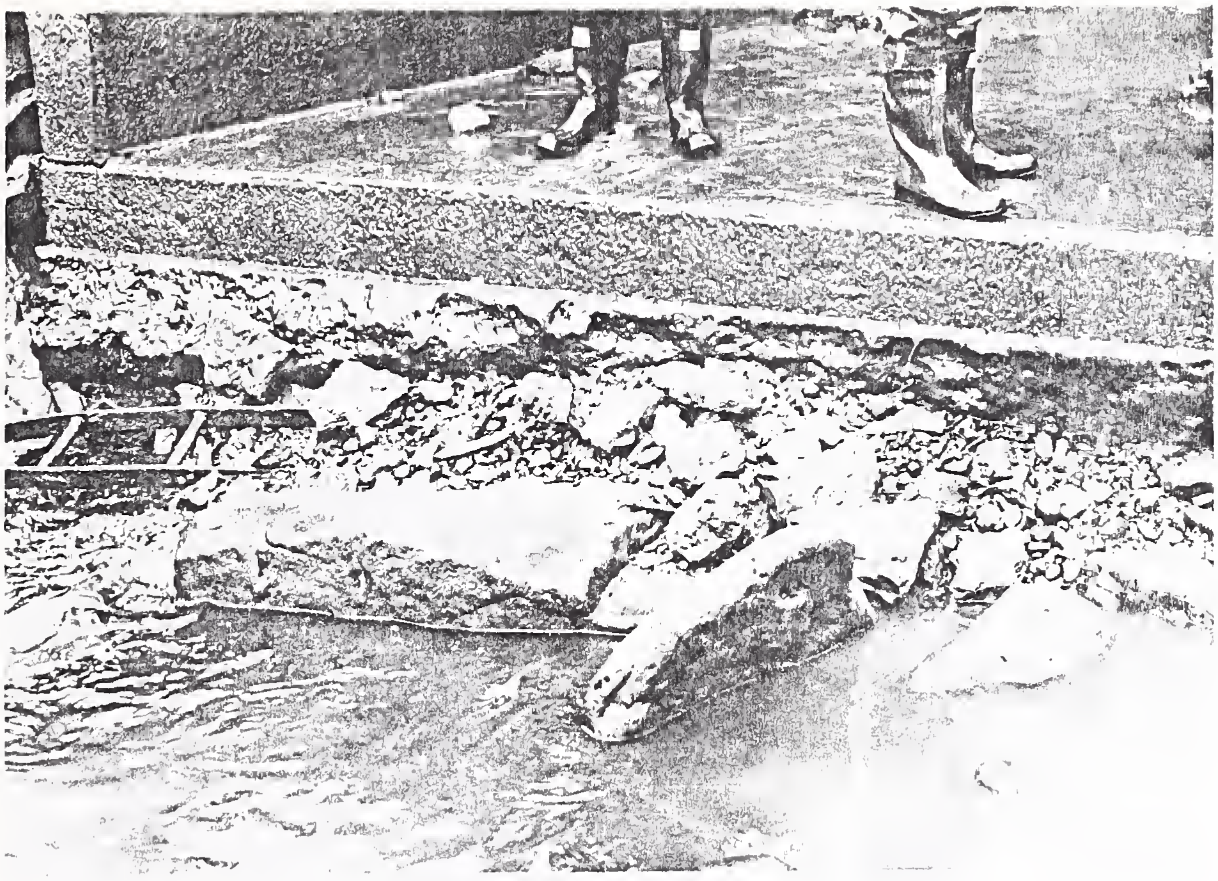


Photo No. 29 - Scour at the Outlet Structure
The outlet structure has been undercut approximately 3 feet in the stilling basin area. The stilling basin has some large riprap, but the mean size is about 12 inches.

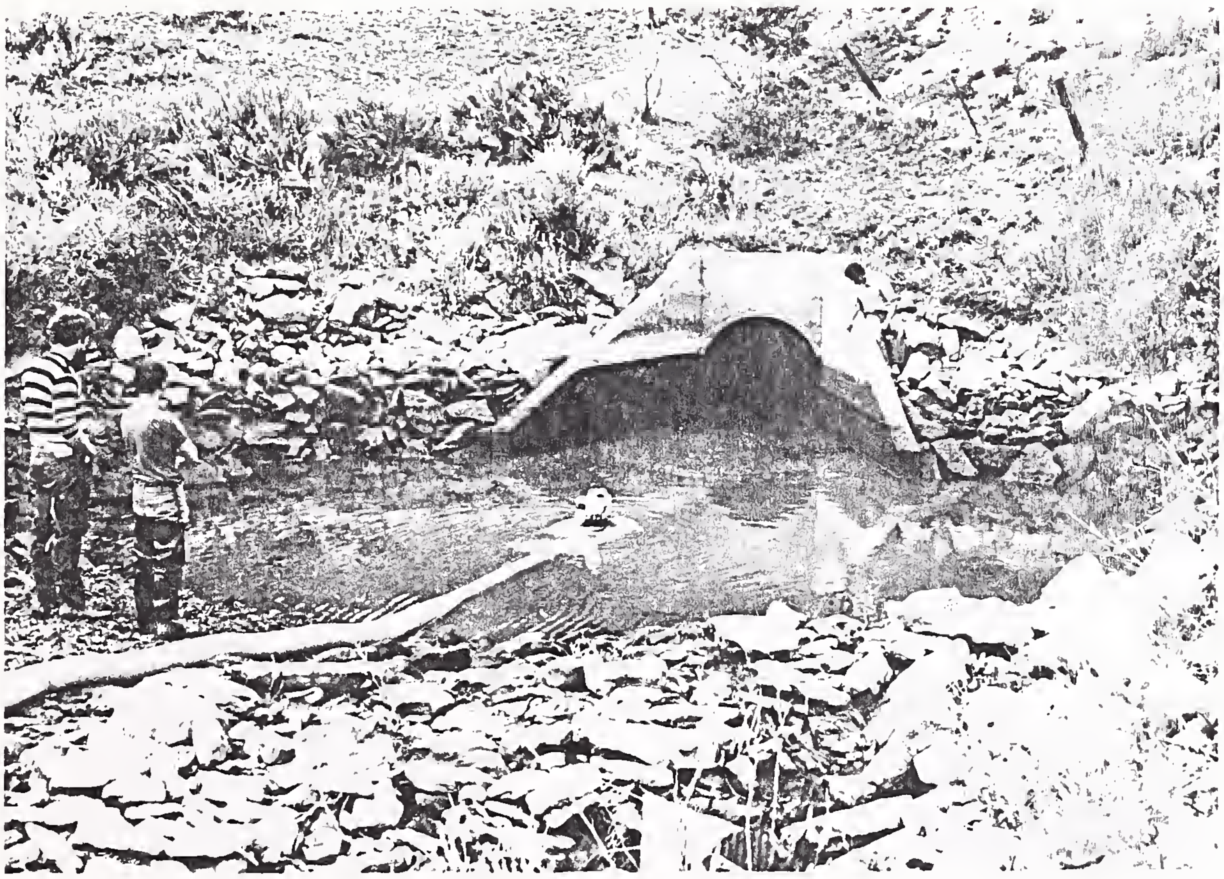


Photo No. 30 - Cutlet Structure
The stilling basin is riprapped; however, the riprap appears small and scattered. The stilling basin is fairly deep due to scour. Notice the normal operating water surface line along the rock banks.

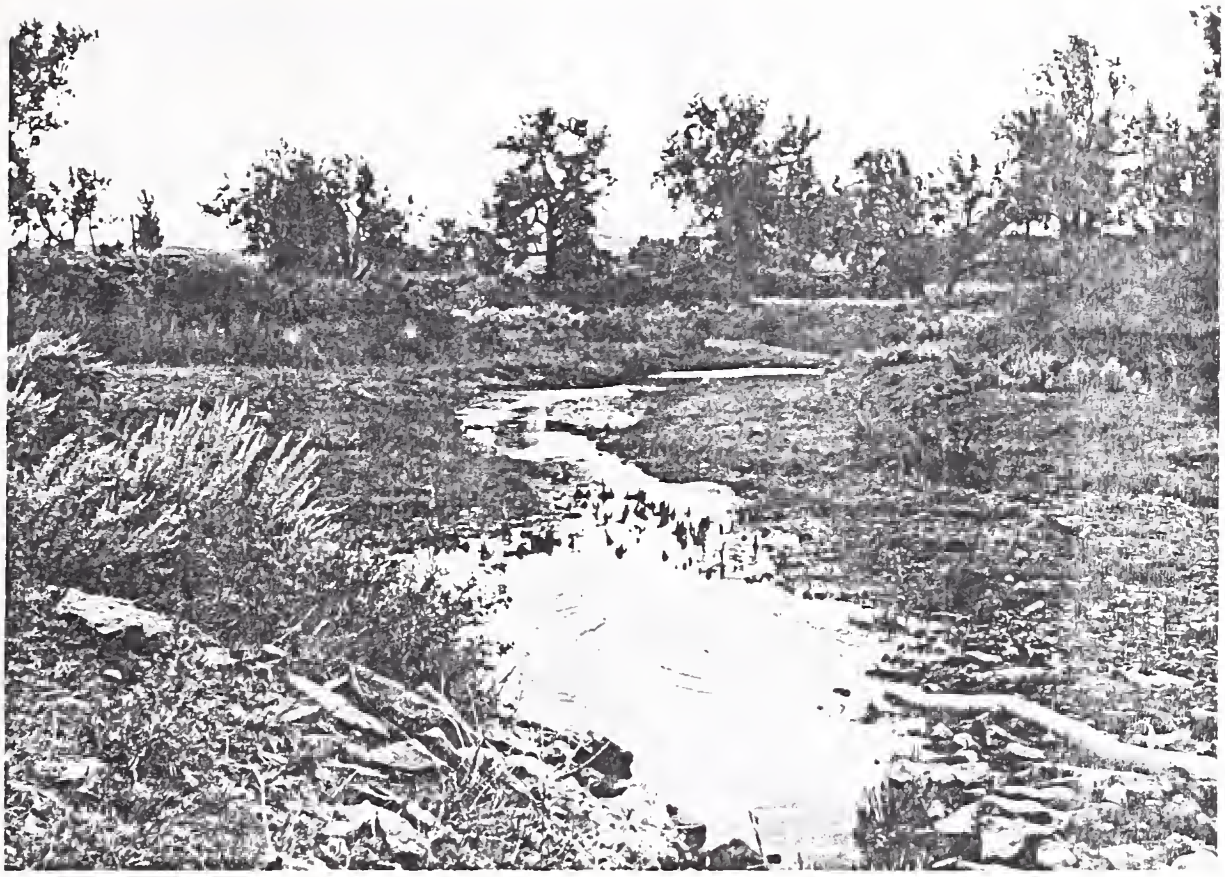


Photo No. 31 - Outlet Return channel
Notice the irregular geometry and small scour hole downstream of riprap terminus (at pump discharge point).

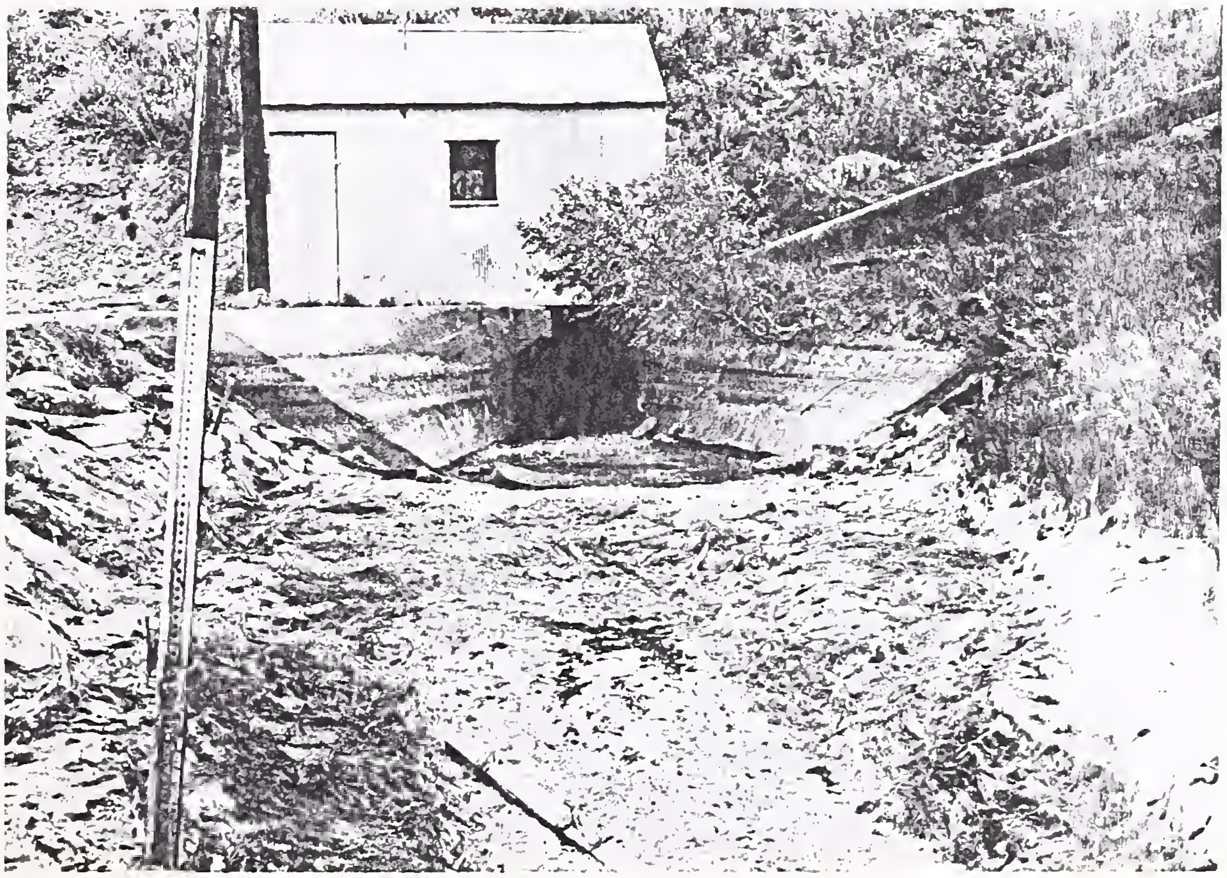


Photo No. 32 - Low-Line Irrigation Canal Outlet
The structure housing the pump and controls is identified in Photo No. 5. Notice the channel is lined for a short distance with concrete, then riprapped. A staff gage is located on the metal post shown in the foreground.

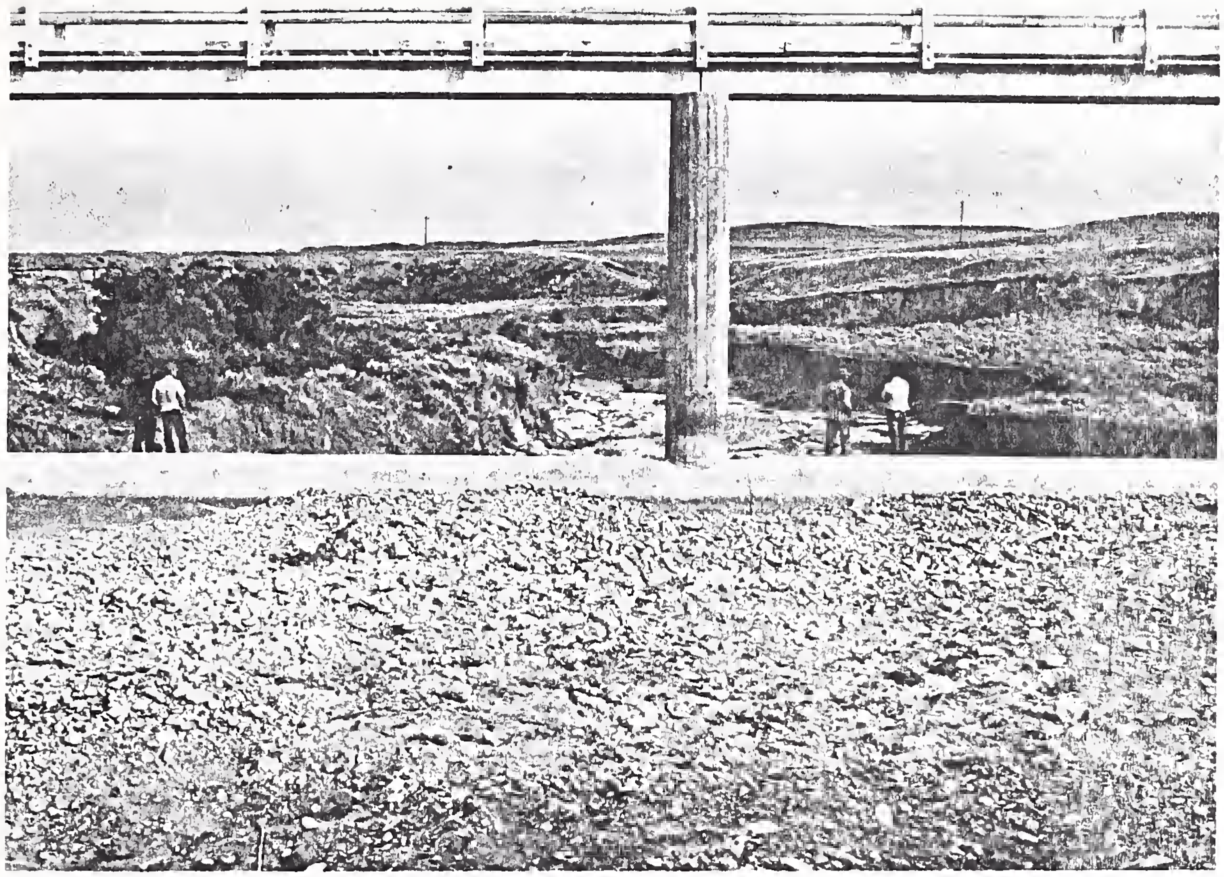


Photo No. 33 - Spillway
Standing in the approach channel looking east over the spillway crest. A foot bridge extends over the spillway. Notice the scour holes on the upstream side of the crest. Also, notice the vertical walls in the return channel.

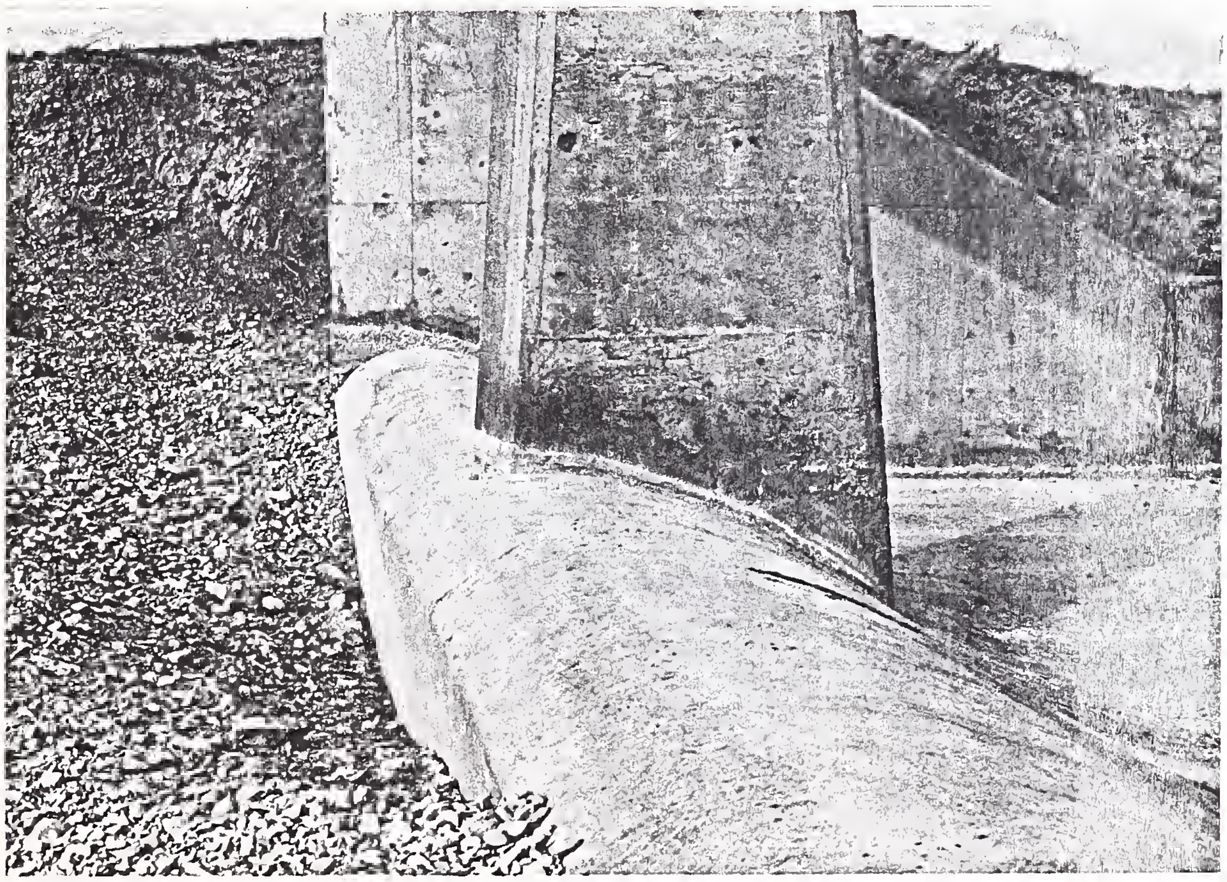


Photo No. 34 - Spillway Crest
The spillway has a concrete ogee crest. Scour holes upstream of the crest are shown. The crest appears in good condition. Some pitting is evident on the pier.

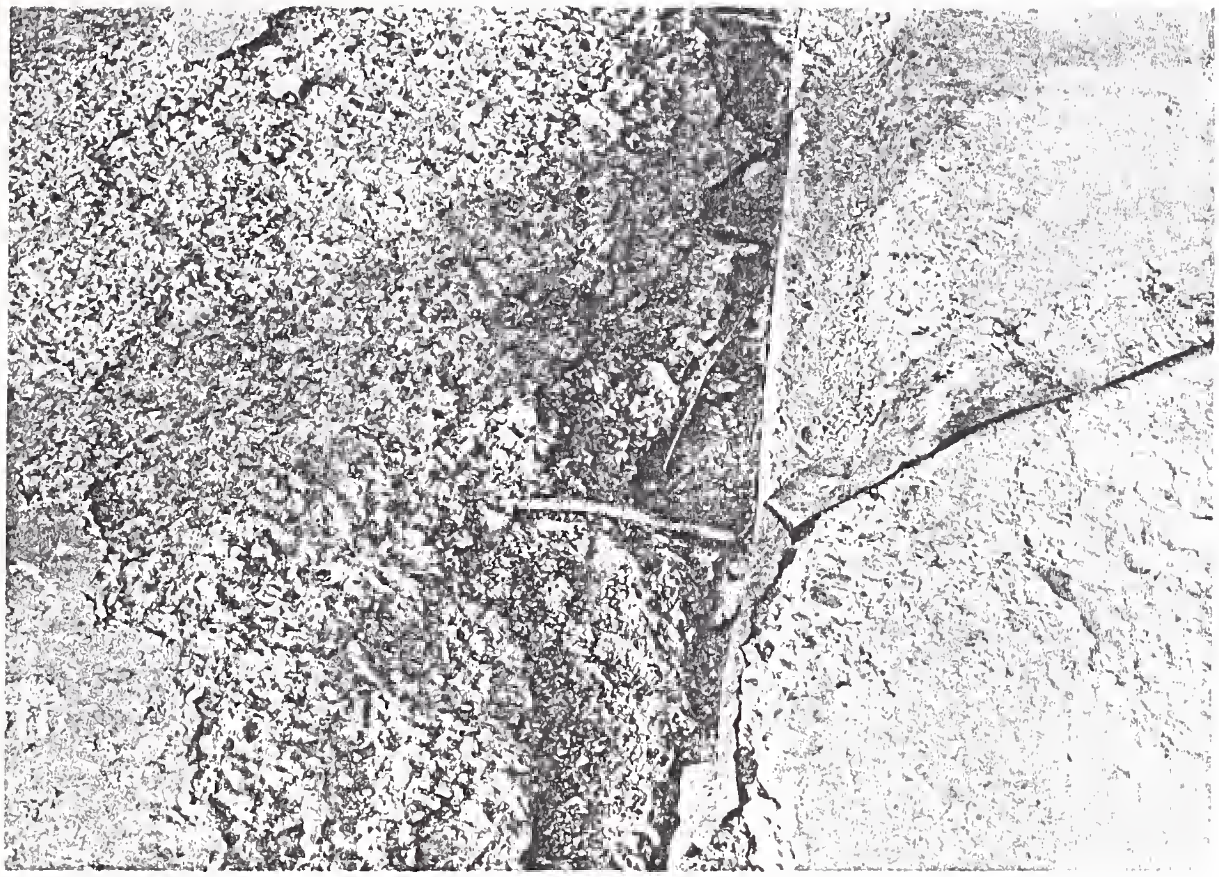


Photo No. 35 - Concrete Deterioration in the Spillway
The concrete in the spillway channel is deteriorating. Rebar is exposed and the metal water-stop has been destroyed.

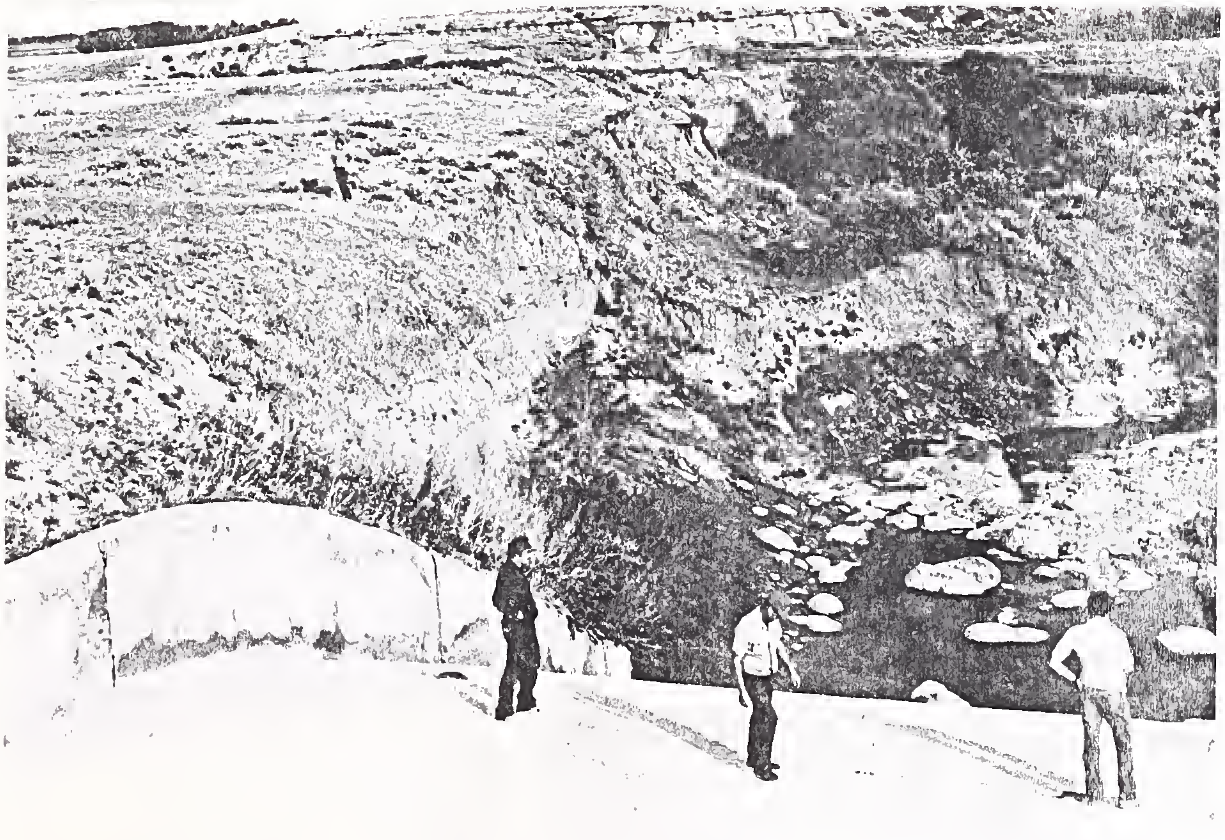


Photo No. 36 - Spillway Chute and Stilling Basin
First of a 3-picture series. The photo shows scour in the stilling basin and d/s channel. Notice the seepage on the channel wall up to about 10 feet above the bottom of the channel.

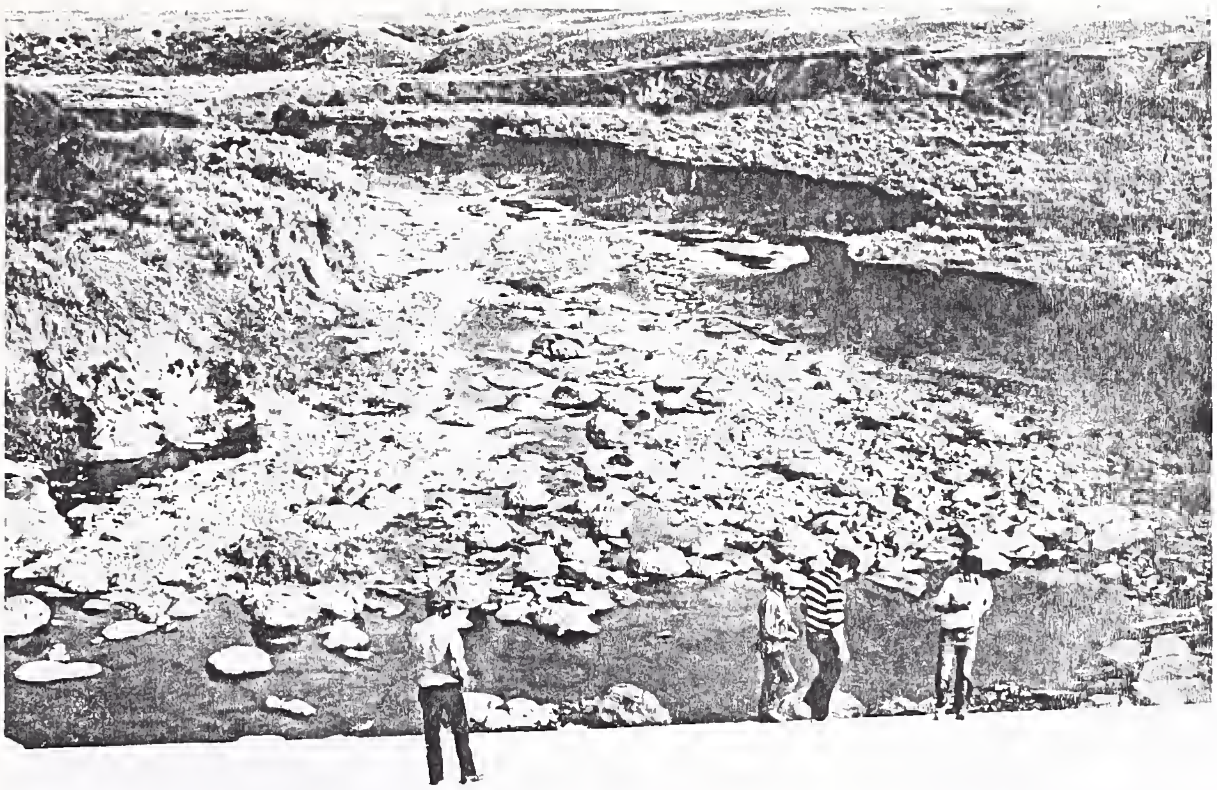


Photo No. 37 - Spillway Chute, Stilling Basin and Return Channel. Second of a 3-picture series. Notice the channel erosion downstream. Riprap below the chute is limited.

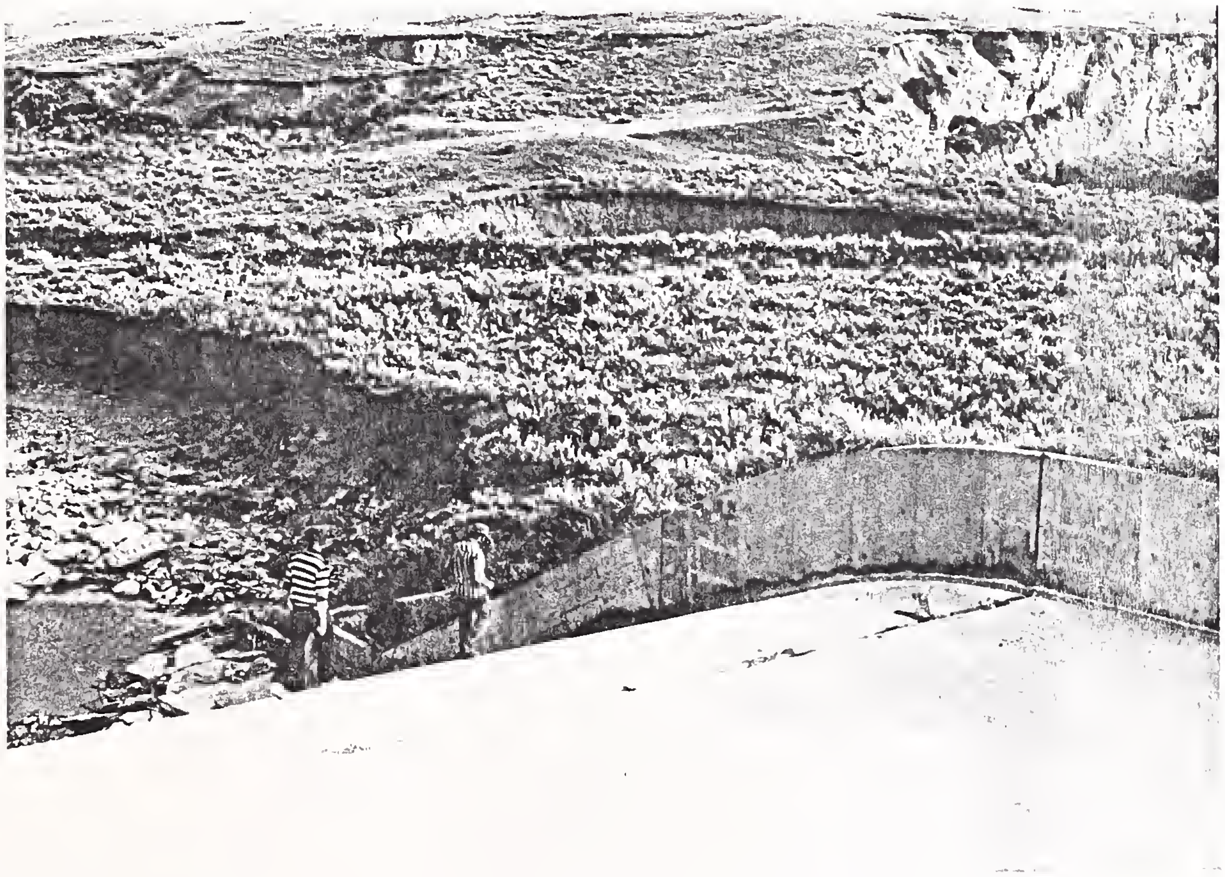


Photo No. 38 - Spillway Chute, Stilling Basin, and Return Channel Third in a 3-picture series. Notice the channel erosion and vertical banks.

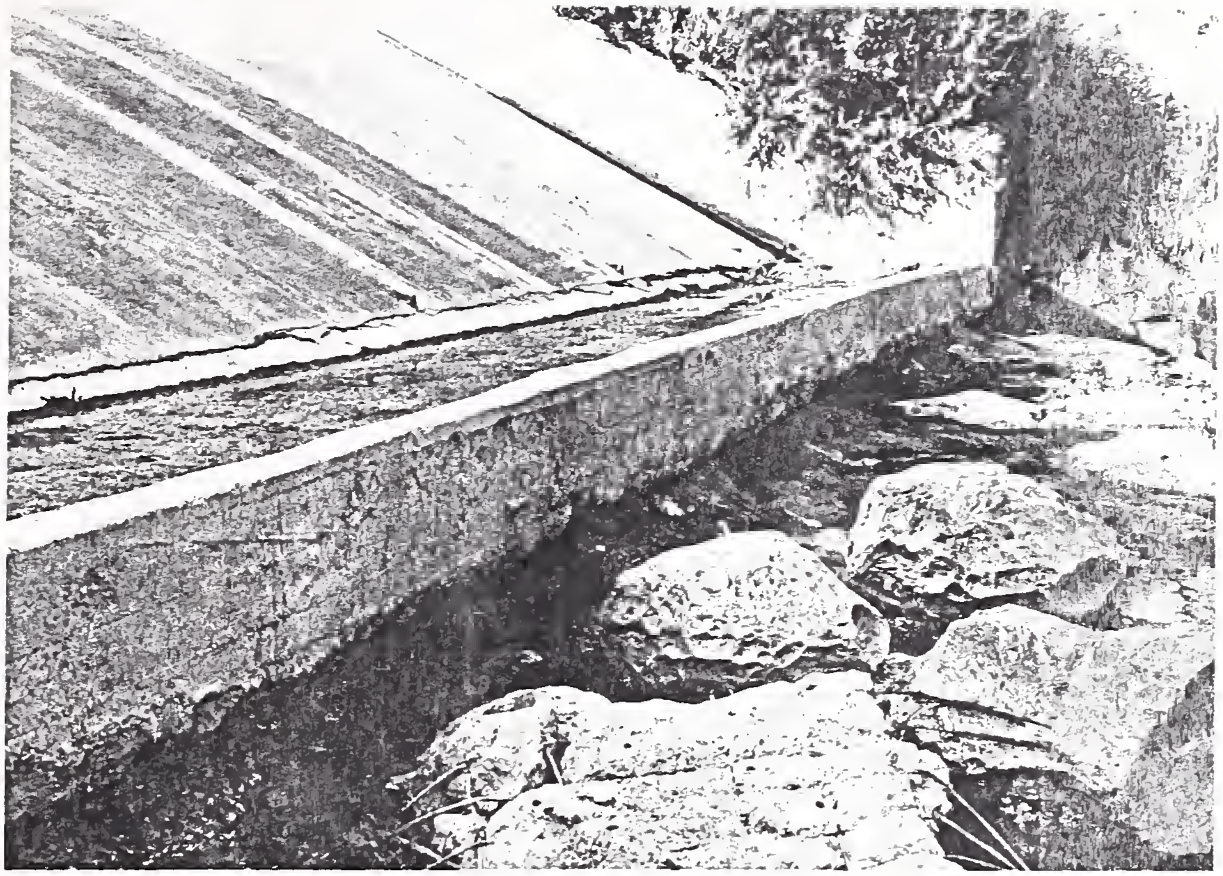


Photo No. 39 - Spillway Lip and Scour
The concrete spillway has been undercut in the stilling basin area. Notice the concrete cracking near the lip, and the ponding water inside the lip or sill.

APPENDIX D

PETROLIA DAM
PROJECT DRAWINGS

Exhibit D1	Construction Plans
Exhibit D2	Dam Crest Profile (Oct. 1979)
Exhibit D3	Measured Embankment Slopes (Sept., 1979)

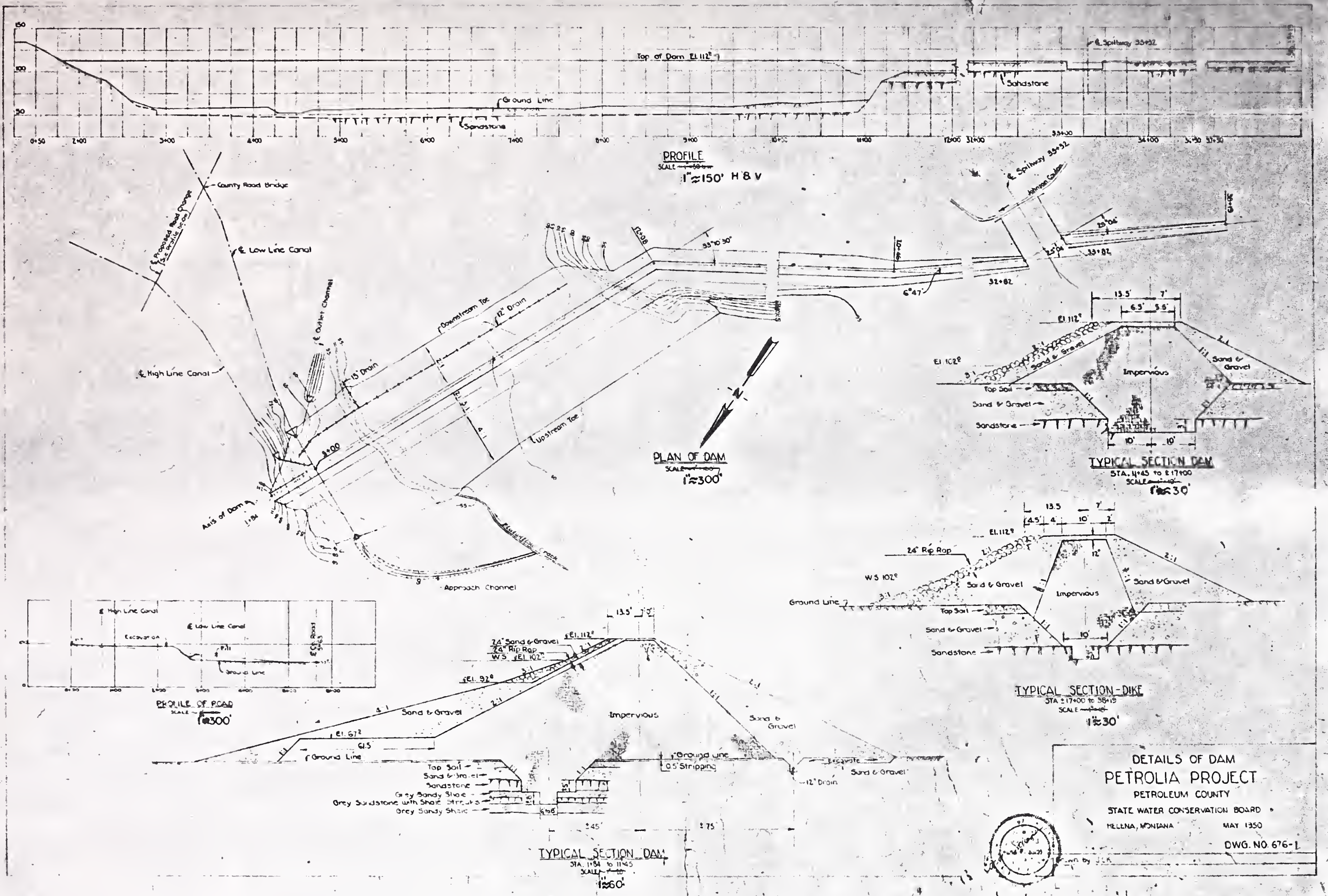
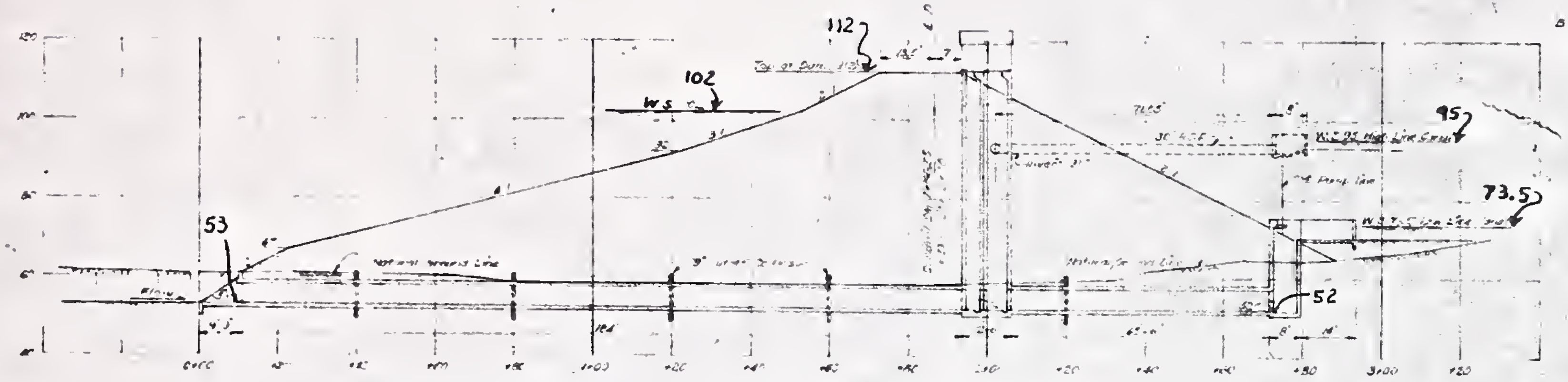
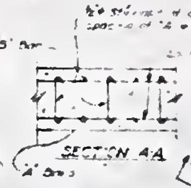


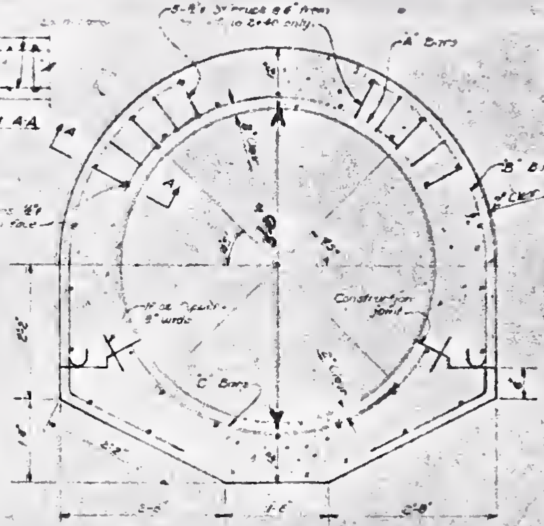
EXHIBIT D1
SHEET NO. 1



Note:
Where conduit does not rest on rock material under conduit shall be removed and replaced with compacted sand, gravel and earth, as directed.

PROFILE OF CONDUIT
1"=60'


SECTION A-A



TYPICAL CONDUIT SECTION
3/4"=3'

SCHEDULE OF CONDUIT REINFORCING

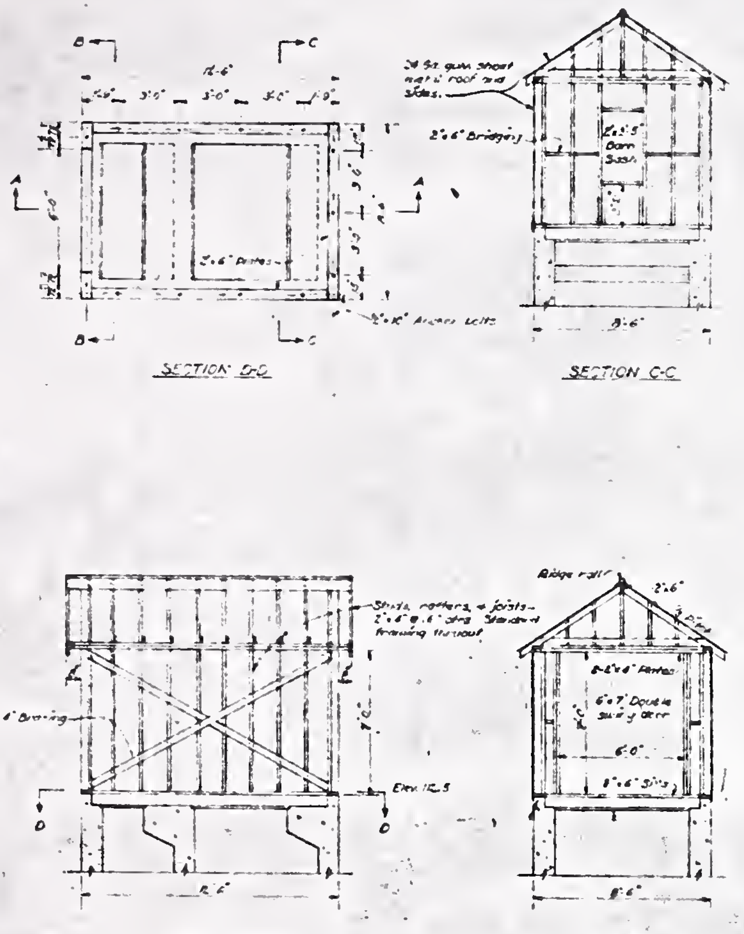
Station	Station	"A" Bars	"B" Bars	"C" Bars
0+11.85	0+30	7/8"φ @ 9"	1"φ @ 10"	3/4"φ @ 6"
0+30	0+70	7/8"φ @ 8"	1"φ @ 9"	3/4"φ @ 7½"
0+70	1+10	7/8"φ @ 7½"	1"φ @ [illegible]	3/4"φ @ [illegible]
1+10	1+50	7/8"φ @ 6"	1"φ @ 5"	3/4"φ @ 6"
1+50	2+16	7/8"φ @ 5"	1"φ @ 5"	3/4"φ @ 5½"
2+16	2+40	7/8"φ @ 6"	1"φ @ 6"	3/4"φ @ 6"
2+40	2+60	7/8"φ @ 8"	1"φ @ 8"	3/4"φ @ 7½"
2+60	2+71.75	5/8"φ @ 10"	1"φ @ 10"	3/4"φ @ 9"

"A" Bars—alternate bars reverse in position as shown except at Sta. 1+10 to Sta. 2+40 where stirrups are required and all bars end 12" below 45° line.

"B" bars—alternate bars with hooks begin at construction joint and end 12" above 45° line, remainder begin near invert and end at 45° line except at Sta 1+10 to Sta. 2+40 where all bars end 12" above 45° line.

"C" Bars—alternate bars reverse in position as shown.

Stirrups required from Sta 1+10 to Sta. 2+40 only.



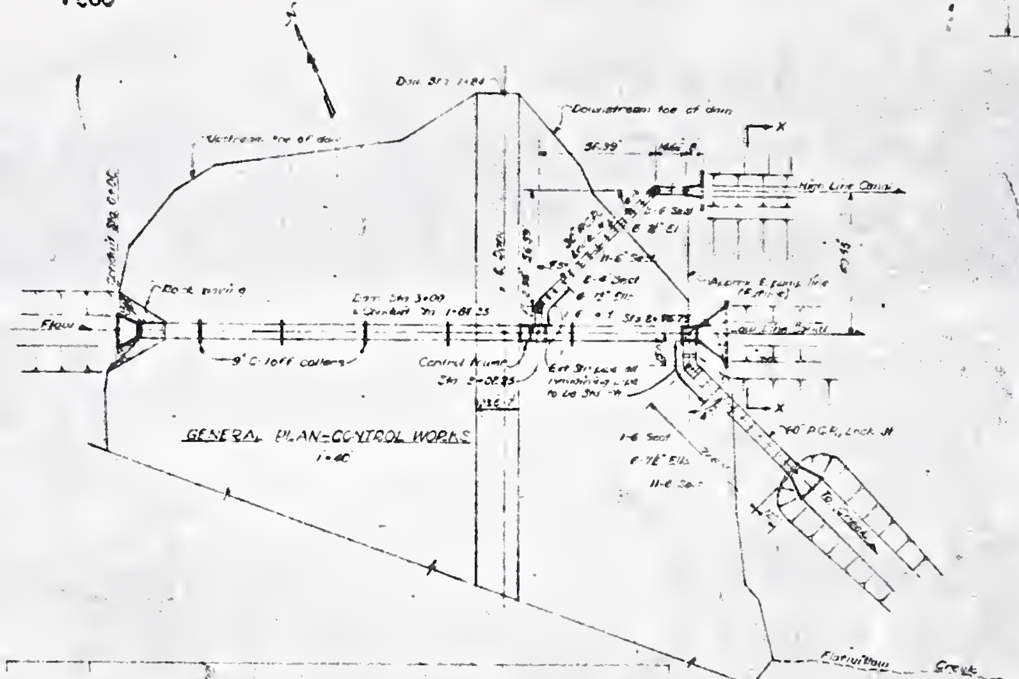
SECTION D-D

SECTION C-C

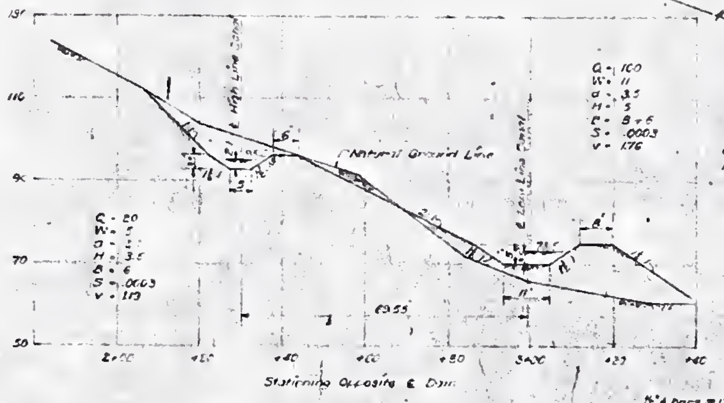
SECTION A-A

SECTION E-E

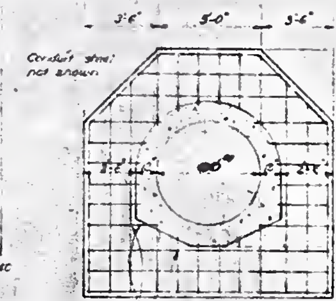
CONTROL HOUSE
1/4"=3'



GENERAL PLAN-CONTROL WORKS
1"=60'



SECTION X-X
1"≈60'

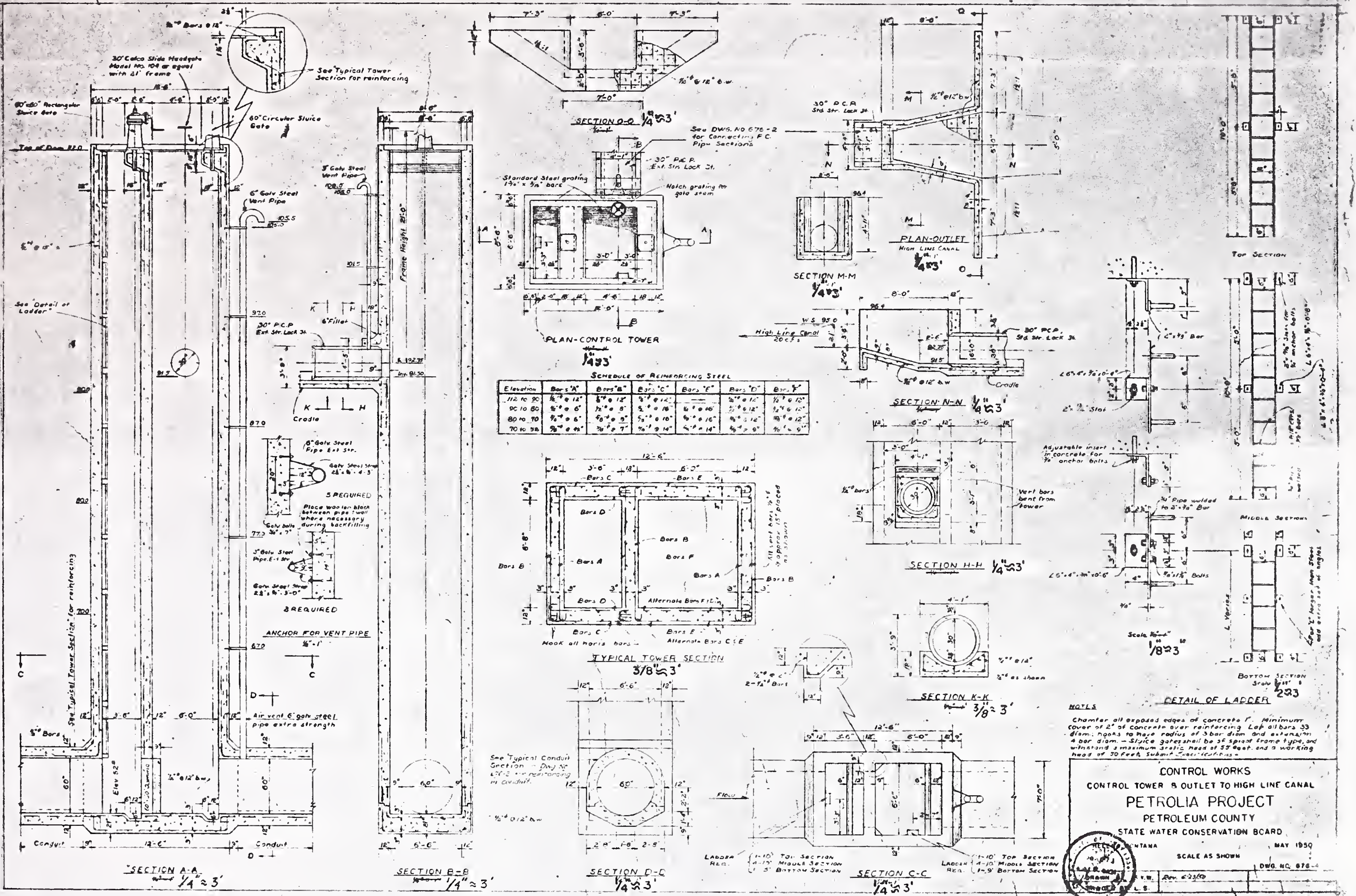


CONDUIT CUTOFF
1/4"≈3'

GENERAL NOTES
Chamfer all exposed edges of concrete 1"
Min cover of 2" of concrete over reinforcing
Lap all bars 33 diam.; hooks to have radius of 3 bar diam. and extension of 4 bar diam.

CONTROL WORKS
GENERAL PLAN—CONTROL HOUSE—CONDUIT SECTION
PETROLIA PROJECT
PETROLEUM COUNTY
STATE WATER CONSERVATION BOARD
HELENA, MONT
MAY 1950
DWG. NO. 676-2
Drawn by
Sgnd

EXHIBIT D1
SHEET NO. 2


SECTION A-A 1/4" = 3'
SECTION B-B 1/4" = 3'
SECTION O-O 1/4" = 3'
PLAN-CONTROL TOWER 1/4" = 3'
PLAN-OUTLET High Line Canal 1/4" = 3'
SECTION M-M 1/4" = 3'
SECTION N-N 1/4" = 3'
SECTION H-H 1/4" = 3'
SECTION K-K 3/8" = 3'
TYPICAL TOWER SECTION 3/8" = 3'
SECTION D-D 1/4" = 3'
SECTION C-C 1/4" = 3'
ANCHOR FOR VENT PIPE
DETAIL OF LADDER
Top Section
Middle Section
Bottom Section
30" Calco Slide Headgate Model No. 104 or equal with 21' frame
60" x 60" Rectangular Sluice Gate
60" Circular Sluice Gate
See Typical Tower Section for reinforcing
6" Galv Steel Vent Pipe
3" Galv Steel Vent Pipe
30" P.C.P. Ext Str. Lock Jt.
Air vent 6" galv steel pipe extra strength
Frame Height 21'-0"
Top of Dam 112.0
See Detail of Ladder
Standard Steel grating
Notch grating for gate stem
See DWG. NO 676-2 for Connecting P.C. Pipe Sections
W.S. 95.0
High Line Canal 20 c.f.s.
Cradle
Conduit
Flow
5 REQUIRED
3 REQUIRED
Place wooden block between pipe & wall where necessary during backfilling
Hook all horiz. bars
Alternate Bars C & E
See Typical Conduit Section - Dwg. 676-2 for reinforcing in conduit.
Adjustable insert in concrete for 3/4" anchor bolts
Vert bars bent from tower

SCHEDULE OF REINFORCING STEEL

Elevation	Bars "A"	Bars "B"	Bars "C"	Bars "E"	Bars "D"	Bars "F"
112 to 90	1/2" @ 12"	5/8" @ 12"	1/2" @ 12"	—	1/2" @ 12"	1/2" @ 12"
90 to 80	5/8" @ 6"	1/2" @ 8"	1/2" @ 18"	1/2" @ 16"	1/2" @ 12"	1/2" @ 12"
80 to 70	7/8" @ 6"	5/8" @ 9"	5/8" @ 10"	5/8" @ 16"	5/8" @ 12"	5/8" @ 12"
70 to 58	7/8" @ 4½"	7/8" @ 7"	7/8" @ 14"	5/8" @ 14"	5/8" @ 9"	7/8" @ 9"

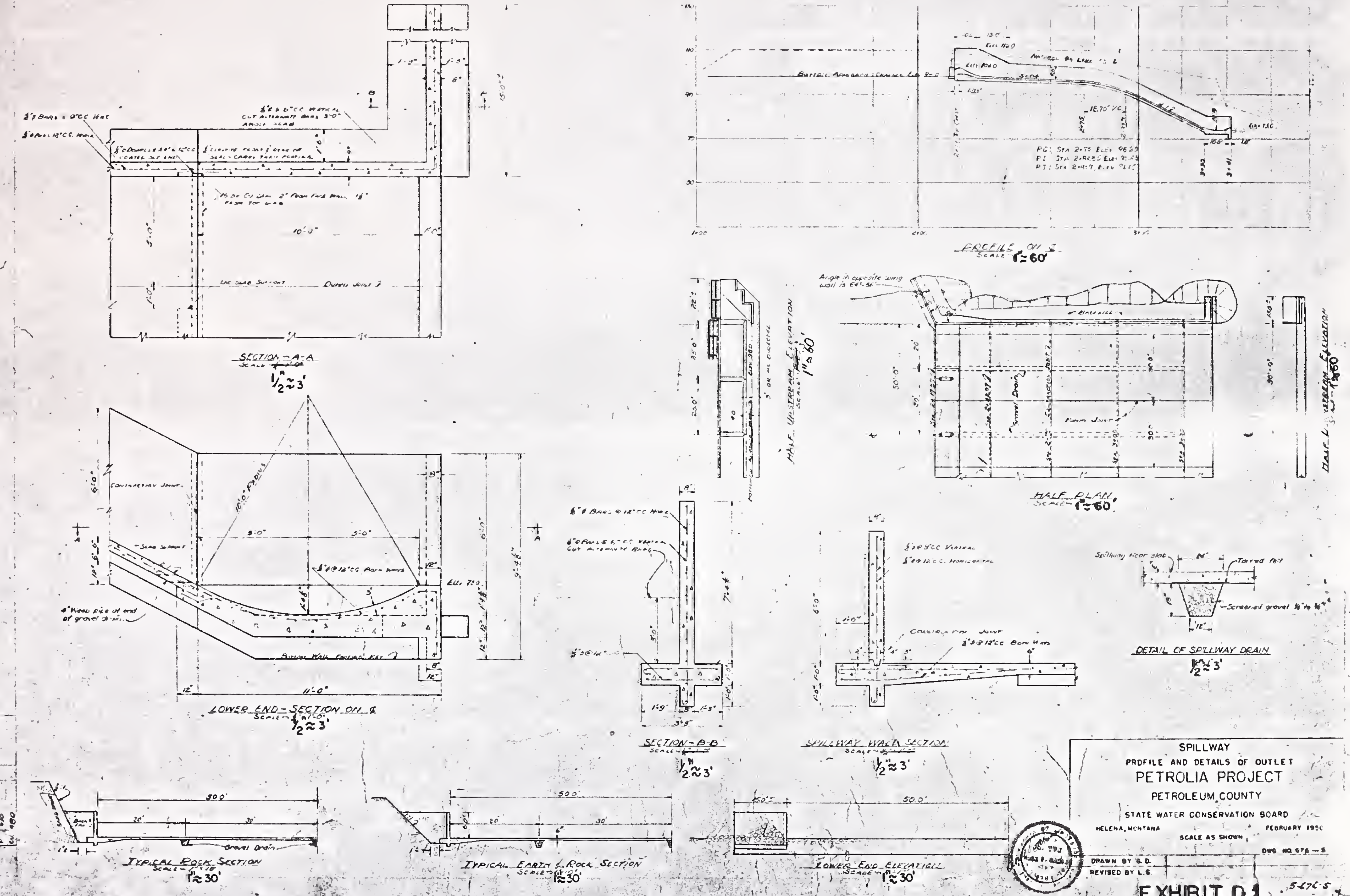
NOTES

Chamfer all exposed edges of concrete 1". Minimum cover of 2" of concrete over reinforcing. Lap all bars 33 diam.; hooks to have radius of 3 bar diam. and extension 4 bar diam. — Sluice gates shall be of spigot frame type, and withstand a maximum static head of 55 feet, and a working head of 30 feet. Submit specifications.

CONTROL WORKS
CONTROL TOWER & OUTLET TO HIGH LINE CANAL
PETROLIA PROJECT
PETROLEUM COUNTY
STATE WATER CONSERVATION BOARD
HELENA, MONTANA
MAY 1950
SCALE AS SHOWN
DWG. NO. 676-4
DRAWN T.W.
CHECKED L.S.
Rev. 6-23-50

EXHIBIT D1
SHEET NO. 3

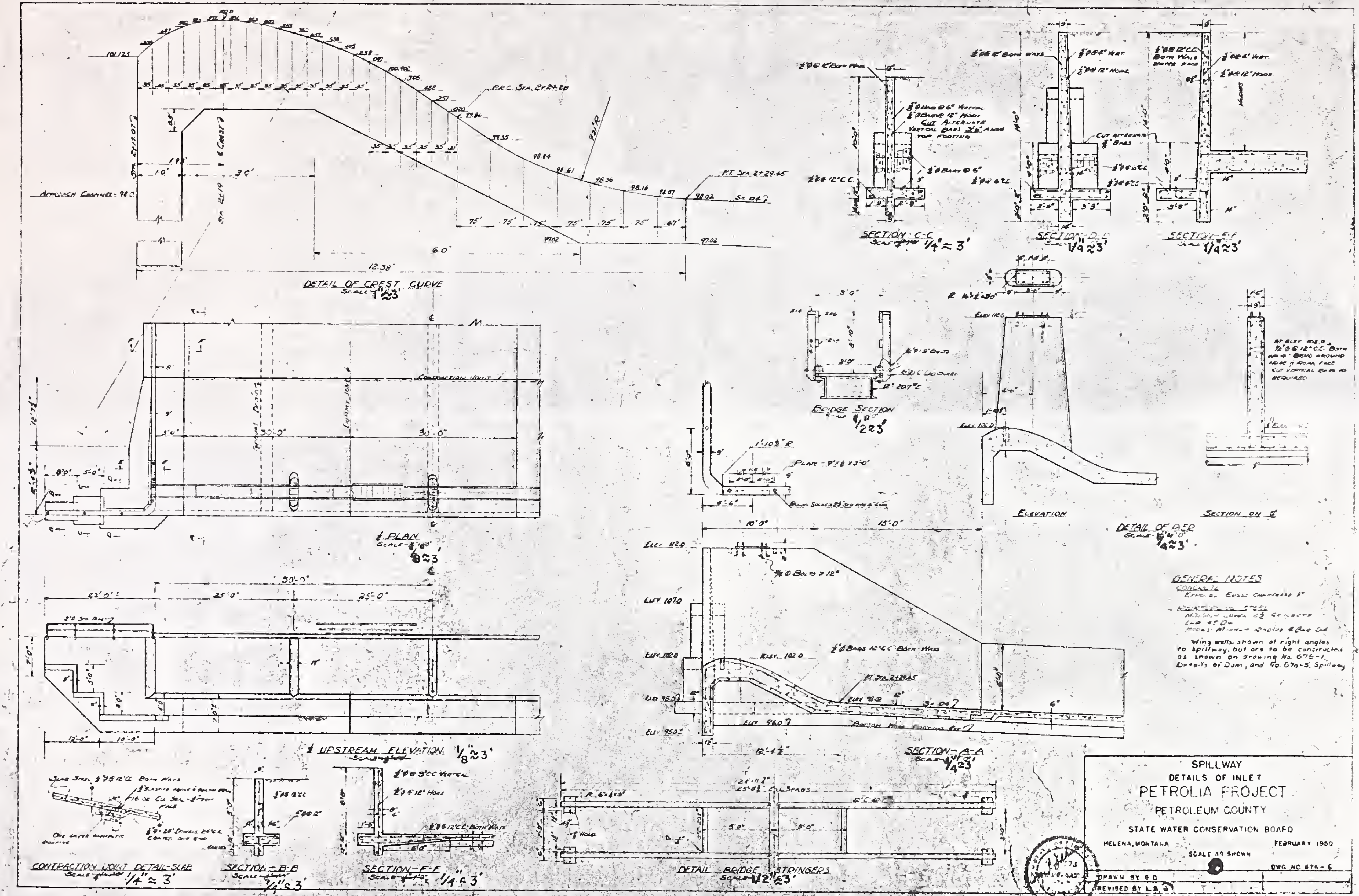
5-676-4



SPILLWAY
PROFILE AND DETAILS OF OUTLET
PETROLIA PROJECT
PETROLEUM COUNTY
STATE WATER CONSERVATION BOARD
HELENA, MONTANA
FEBRUARY 1950
SCALE AS SHOWN
DWG NO 676 – 5
DRAWN BY G. D.
REVISED BY L. S.

EXHIBIT D1
SHEET NO. 4

5-676-5



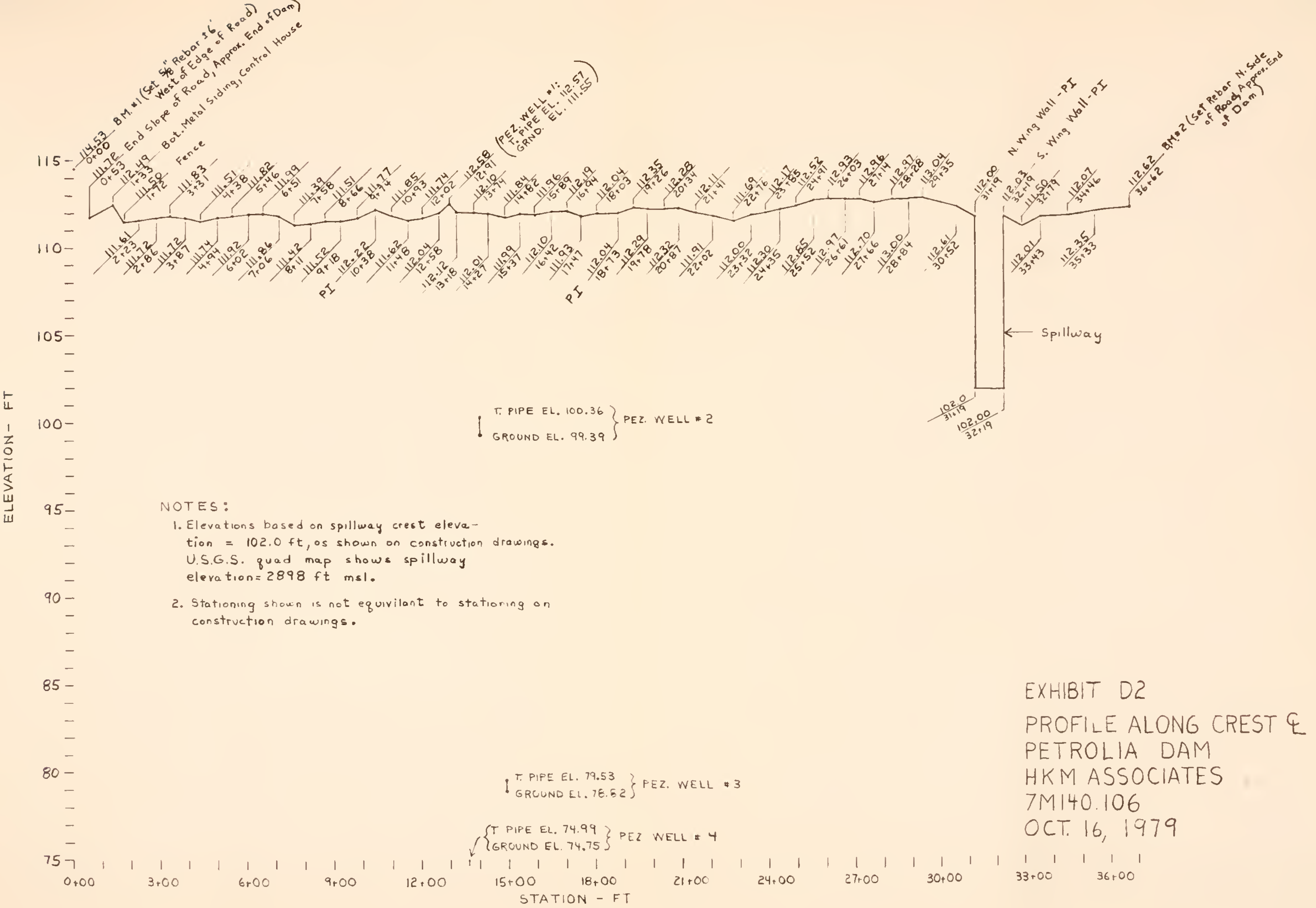
GENERAL NOTES
CONCRETE
Exposed Edges Chamfered 1"
REINFORCING STEEL
Minimum Cover 2½ Concrete
Lap 45 Dia.
Hooks Minimum Radius 4 Bar Dia.

Wing walls shown at right angles to spillway, but are to be constructed as shown on drawing No. 676-1, Details of Dam, and No. 676-5, Spillway

SPILLWAY
DETAILS OF INLET
PETROLIA PROJECT
PETROLEUM COUNTY
STATE WATER CONSERVATION BOARD
HELENA, MONTANA
FEBRUARY 1950
SCALE AS SHOWN
DWG. NO. 676-6
DRAWN BY G.D.
REVISED BY L.S.

EXHIBIT D1
SHEET NO. 5

5-676-6



NOTES:

1. Elevations based on spillway crest elevation = 102.0 ft, as shown on construction drawings. U.S.G.S. quad map shows spillway elevation = 2898 ft msl.
2. Stationing shown is not equivilant to stationing on construction drawings.

EXHIBIT D2
PROFILE ALONG CREST ℄
PETROLIA DAM
HKM ASSOCIATES
7M140.106
OCT. 16, 1979

HKM ASSOCIATES COMPUTATION SHEET

By DRC	**Date** 11-5-79	**Project No.** 7M140.106	**Sheet** 1 **of** 2
Chkd by	**Date**	**Name** PETROLIA DAM	
Details Station-Elevation Tabulation	**Feature** Crest Survey		

Station (ft)	Elevation (ft)	Description
0+00	114.53	B.M.#1-5/8"Rebar Set ±6' west of access road in line with the south track of road that runs east
0+53	111.72	End slope of road, Begin dam
1+33	112.49	Bot. of metal siding, S.W. corner of control house
1+72	111.50	Road at fence
2+23	111.61	
2+86	111.72	
3+31	111.83	
3+87	111.72	
4+38	111.51	
4+94	111.74	
5+46	111.82	
6+02	111.92	
6+51	111.99	
7+06	111.86	
7+58	111.39	
8+11	111.42	
8+66	111.51	
9+18	111.52	
9+74	111.77	
10+38	112.22	PI - Turning point of dam axis
10+93	111.85	
11+48	111.62	
12+02	111.74	
12+58	112.04	
12+91	112.58	
13+18	112.12	
13+74	112.10	
14+27	112.01	
14+82	111.84	
15+37	111.99	
15+89	111.96	
16+42	112.10	
16+94	112.19	
17+47	111.93	

EXHIBIT D2

HKM ASSOCIATES COMPUTATION SHEET

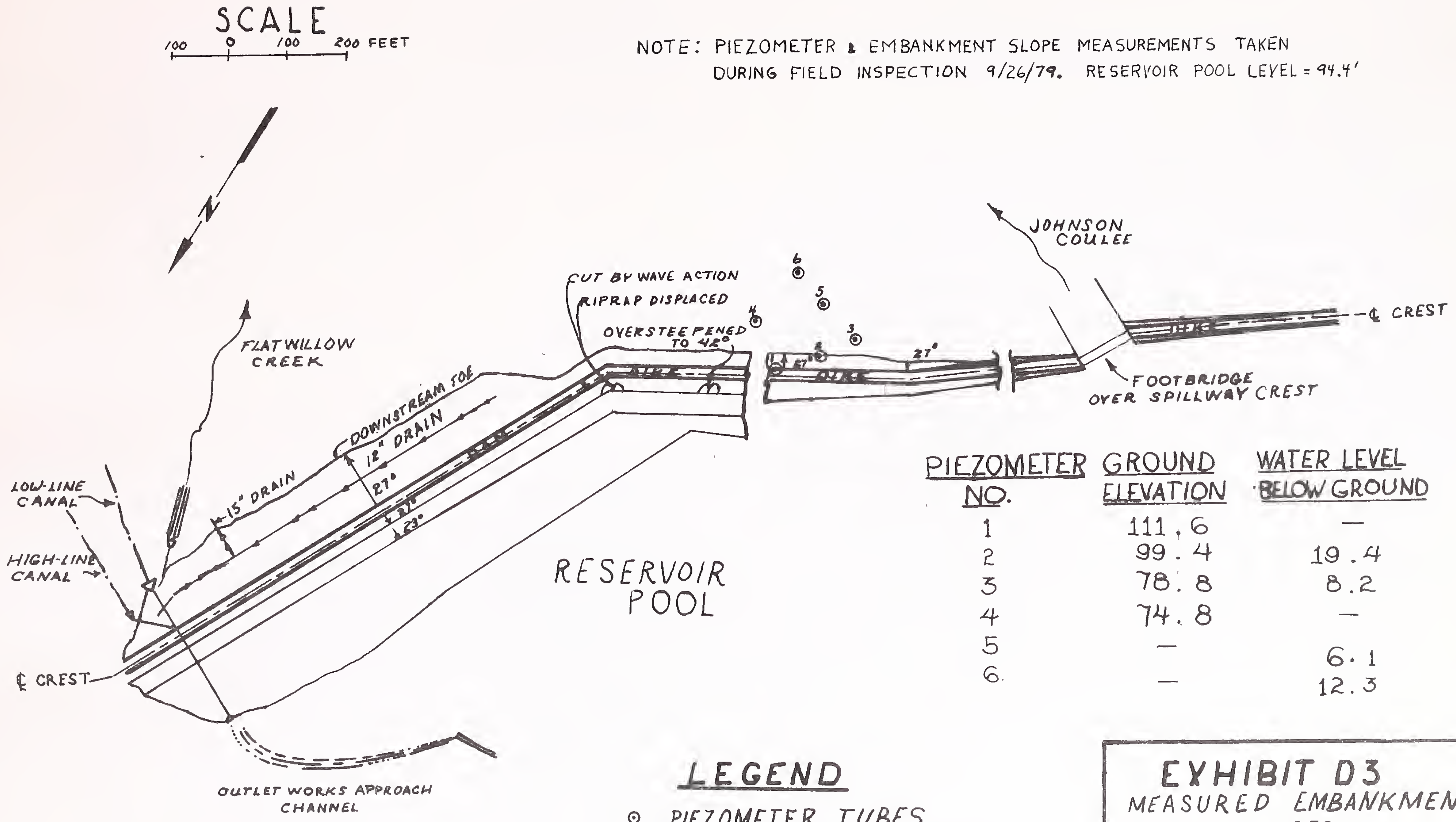
By DRC	**Date** 11-5-79	**Project No.** 7M140.106	**Sheet** 2 **of** 2
Chkd by	**Date**	**Name** PETROLIA DAM	

Details Station-Elevation Tabulation	**Feature** Crest Survey

Station (ft)	Elevation (ft)	Description
18+03	112.04	
18+73	112.04	PI – Turning point of dam axis
19+26	112.35	
19+78	112.29	
20+34	112.28	
20+87	112.32	
21+41	112.11	
22+02	111.91	
22+76	111.69	
23+32	112.00	
23+85	112.17	
24+35	112.30	
24+91	112.52	
25+52	112.85	
26+03	112.93	
26+61	112.97	
27+14	112.96	
27+66	112.70	
28+28	112.97	
28+84	113.00	
29+35	113.04	
30+52	112.61	
31+19	112.00	N. wing wall, spillway; PI – Turning point, dam axis
32+19	102.00	Spillway crest, S. Side
32+19	112.03	S. wing wall, spillway; PI – Turning point, dam axis
32+79	111.50	
33+43	112.01	
34+46	112.07	
35+33	112.35	
36+62	112.62	B.M. # 2 – 5/8" Rebar set at end of dam fill, ±5' from edge of road, on dam axis ℄

EXHIBIT D2

NOTE: PIEZOMETER & EMBANKMENT SLOPE MEASUREMENTS TAKEN
DURING FIELD INSPECTION 9/26/79. RESERVOIR POOL LEVEL = 94.4'



PIEZOMETER NO.	GROUND ELEVATION	WATER LEVEL BELOW GROUND
1	111.6	—
2	99.4	19.4
3	78.8	8.2
4	74.8	—
5	—	6.1
6	—	12.3

LEGEND

⊙ PIEZOMETER TUBES
↓27° MEASURED EMBANKMENT SLOPES

EXHIBIT D3
MEASURED EMBANKMENT SLOPES

PETROLIA DAM

HKM Assoc. 7M140.106

APPENDIX E

PETROLIA DAM
ENGINEERING DATA

Exhibit E1	Reservoir Elevation - Capacity Table
Exhibit E2	Reservoir Elevation - Capacity Curve
Exhibit E3	Discharge Rating Table
Exhibit E4	Discharge Rating Curves

EXHIBIT E1

RESERVOIR ELEVATION-CAPACITY TABLE

PETROLIA DAM

Reservoir Elevation-ft	Reservoir Capacity-acre-ft	
	Dead Storage	Active Storage
60	0	0
70	370	0
75		420
80		1,166
85		2,307
90		3,840
95		5,822
100		8,182
102		9,192
109		12,528
112		13,966

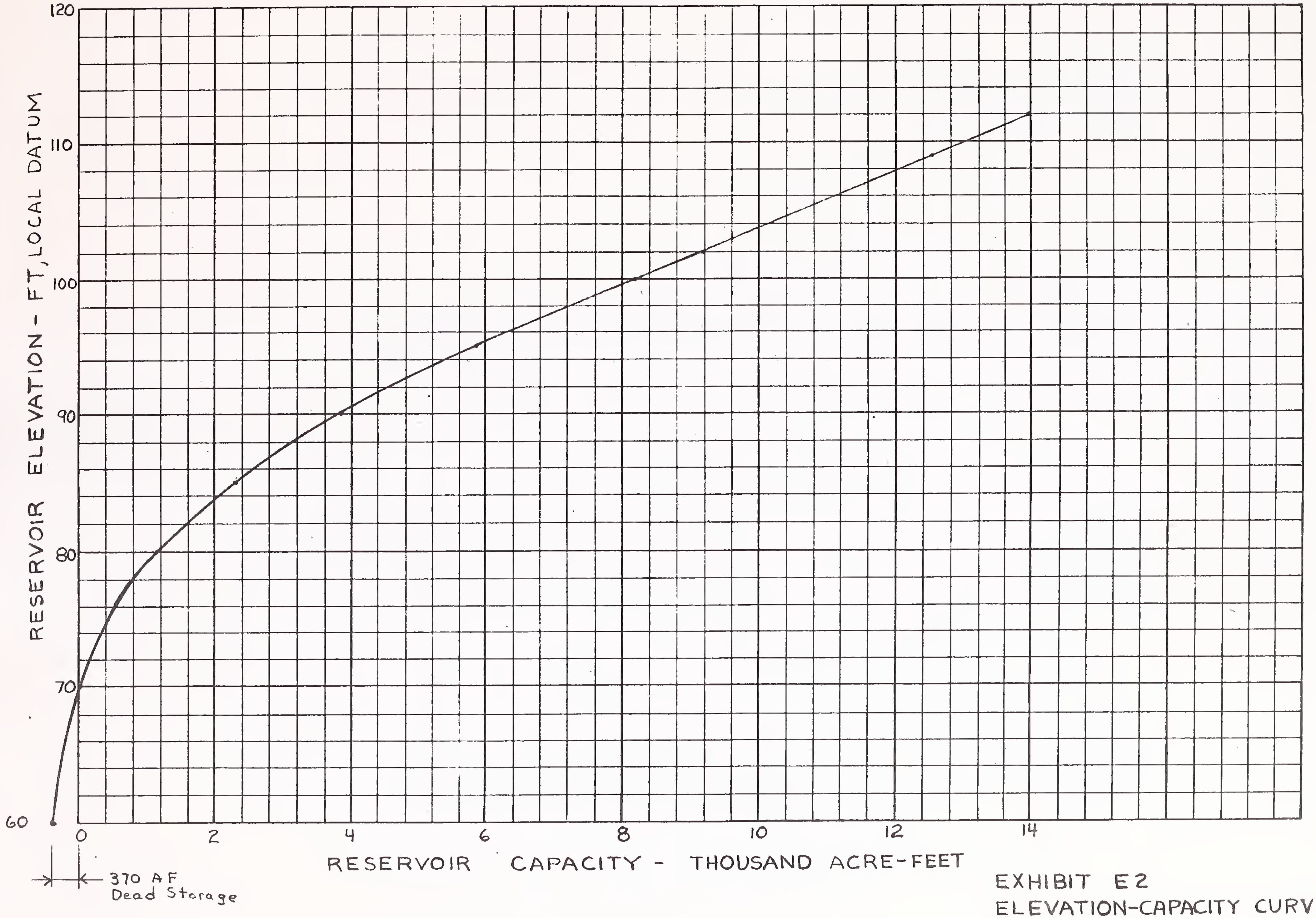


EXHIBIT E2
ELEVATION-CAPACITY CURVE
PETROLIA DAM

EXHIBIT E3
DISCHARGE RATING TABLE
PETROLIA DAM

Reservoir Elevation (ft)	Outlet Works Discharge (cfs)	Spillway Discharge (cfs)	Total Discharge (cfs)
53	0		
60	110		
70	230		
80	305		
90	365		
100	415		
101	420		
102	425	0	425
103	430	290	720
104	435	840	1,275
105	440	1,600	2,040
106	445	2,600	3,045
107	450	3,700	4,150
108	455	4,900	5,355
109	460	6,300	6,760
110	460	7,800	8,260
111	465	9,500	9,965
111.4	470	10,200	10,670
112	470	11,300	11,770

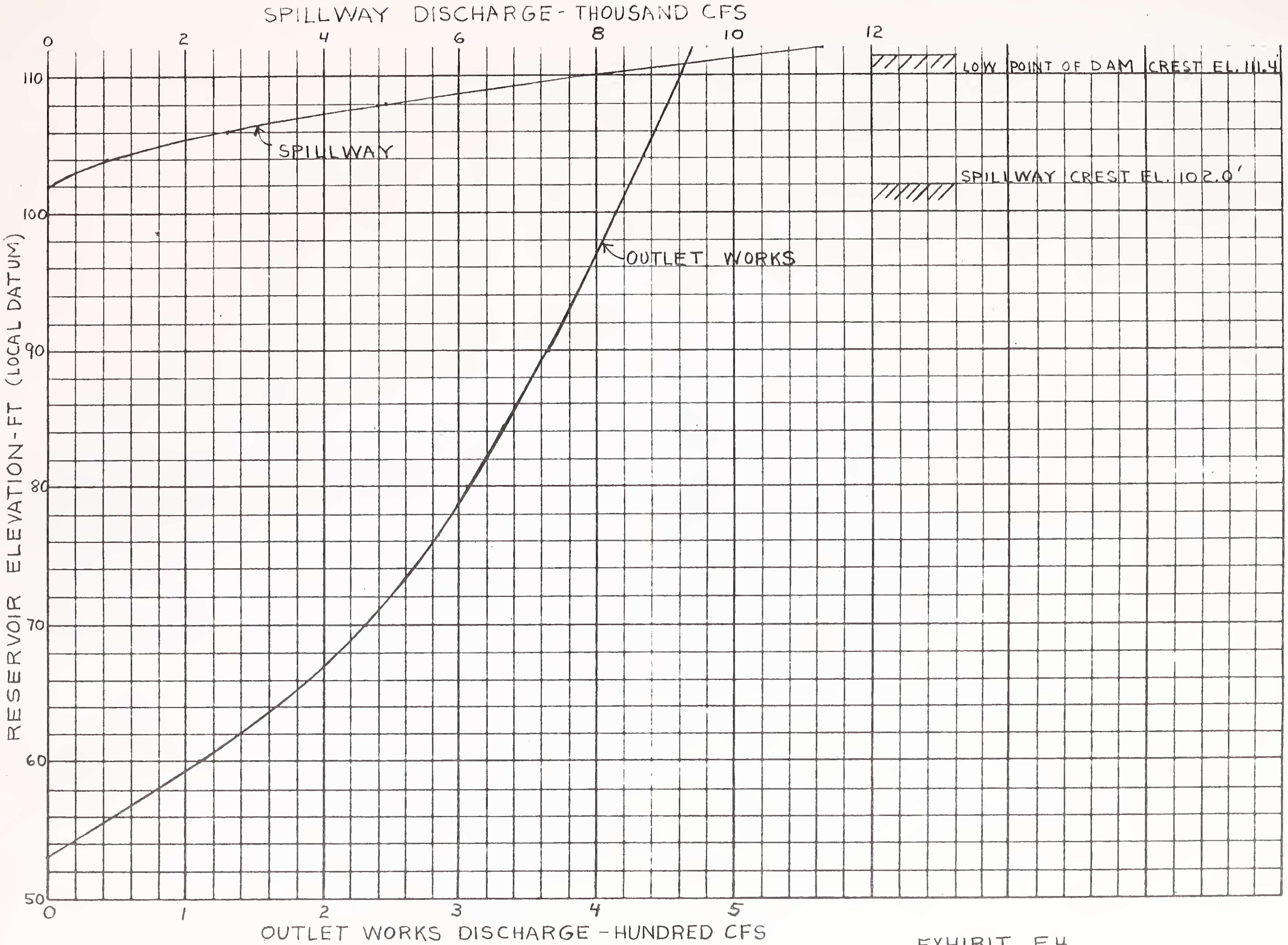


EXHIBIT E4
DISCHARGE RATING CURVES
PETROLIA DAM

APPENDIX F

PETROLIA DAM
CORRESPONDENCE

DEPARTMENT OF NATURAL RESOURCES
AND CONSERVATION
WATER RESOURCES DIVISION

THOMAS L. JUDGE, GOVERNOR
32 SOUTH EWING
STATE OF MONTANA
(406) 449-2872
HELENA, MONTANA 59601

October 29, 1980

Department of the Army
Seattle District, Corps of Engineers
P.O. Box C-3755
Seattle, WA 98124

Attn: Ralph Morrison

Re: HKM Associates Dam Safety Inspection Report on Petrolia Dam (MT-8)

Dear Ralph:

We have reviewed the above referenced final draft report. We concur with the findings and recommendations and find that is satisfies the criteria of the Phase I report.

Minor editorial comments have been discussed with your staff, and we understand these will be incorporated in the final report.

Enclosed is a copy of the letter that the Petrolia Water Users Association sent to us with their comments on the inspection report.

At the present time, the reservoir is being drained so that the inlet and upper portion of the outlet pipe can be inspected during the week of November 3, 1980. HKM Associates have been informed of the inspection and may send a represenative to attend the inspections, if not they will be informed of the findings.

Thank you for this opportunity to review and comment on the final draft report on Petrolia Dam.

Sincerely,

Arthur D Taylor

Arthur D. Taylor
Dam Safety Engineer
Engineering Bureau
(406) 449-2864

AT/lj

MEMBER OF MONTANA WATER USERS ASSOCIATION

PETROLIA WATER USERS ASSOCIATION

PETROLEUM COUNTY

Winnett, Montana 59087

RECEIVED
[illegible] 1980
[illegible] NATURAL
[illegible] CONSERVATION

October 25, 1980

Arthur D. Taylor
Dam Safety Engineer
Engineering Bureau
Helena, Montana 59601

Dear Sir:

The Petrolia Water Users Association Board of Directors have reviewed the Phase I Inspection report and this is what has been done.

On or about the 20th of May 1970 100 yards of rip rap was put on the south end of the dam.

In October of 1979 we opened up the spillway and put approximately 40 yards of gravel under the apron, then sealed and grouted the expansion joints. We also dug down ahead of the spillway crest to see if the water was getting through and under, but there was no sign of the same, then the spillway throat was leveled. There was approximately 20 yards of rip rap put in by the north wing of the spillway in May of 79.

The lower creek turnout gate control pedestal was repaired in July of 80.

The Petrolia Water Users Board of Directors disagrees with Phase I inspection report prepared by HKM Associates in that if there was a spillway of the proportion that they say is needed, there would be a hazard to the people living below the dam the same as they talk of no.

Petrolia Water Users Association
by Orene Barber
Sec.

MEMORANDUM December 5, 1980

TO: The Record

FROM: Art Taylor *at*
Dam Safety Engineer
Engineering Bureau

RE: Dam inspection at Petrolia Dam (MT-8) on November 12, 1980.

Participants

Glen McDonald - DNRC
Art Taylor - DNRC
Jerry Bohn - Petrolia Water User
Clinton Hassett - Petrolia Water User

On November 12, 1980, we tried to inspect the upper end of the conduit at Petrolia, but we could not enter the conduit due to high flows; about 70 to 100 cfs. Approximate water surface when we arrived was at elevation 60.00 local datum. All elevations are relative to the top of concrete in the gate house being 112.49. The top of the trash rack is at about elevation 59.63. The invert of the inlet is at about elevation 51.88. This compares with the plans showing the top of the trash rack at elevation 59.75 and the invert at elevation 53.

Upstream cross-sections were developed for a section along the approximate axis of the pipe and for a section near the fence, which was assumed to be the typical section. (See attached). There are a number of scarfs on the upstream face caused by wave erosion, especially near the inlet area, which may distort the sections somewhat. (See Photo 1, 2).

Shown of the cross sections of the upstream face, the riprap is over steepened and should be re-shaped and the riprap properly bedded.

A line of levels was also run from the gate house to the outlet with the top of the head wall being elevation 59.16 and the top of the silt in the invert being at elevation 52.55. The plans call for the top of the head wall to be elevation 59.5 and the invert to be at elevation 52.0.

When we arrived at the dam we observed that there was water still covering the outlet and upon examination it was observed that the trash rack was plugged with debris with the exception of about one square foot of surface area. Attempts to block this area so that the water could be stopped from entering the pipe failed. We proceeded to clean the debris from the trash rack and remove debris to about elevation 54.0 which is below the trash rack crossbeam.

Flows seemed to decrease some so that about the upper half of the pipe was visible. The portion of the pipe and inlet that were visible appeared to be in good physical condition with only minor rust and corrosion visible.

The inlet canal, from the original streambed, to the inlet structure, was visible but became well defined once the trash rack had been partially cleaned and the silt that had built up in the canal started moving through the outlet.

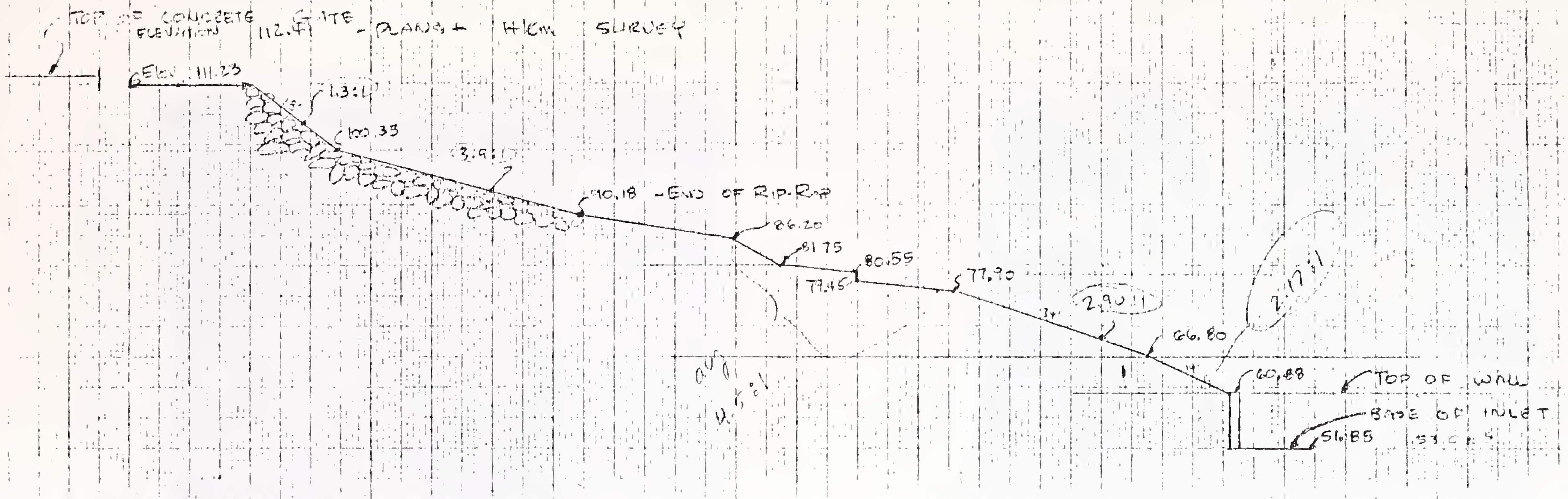
The operating gate in the lower tower binds and is very difficult to operate. There are 3 or 4 bends in the stem which constricts operation. We were only able to open the gate 2½ to 3 feet out of a possible 5 feet. This condition should be corrected.

Recommendations

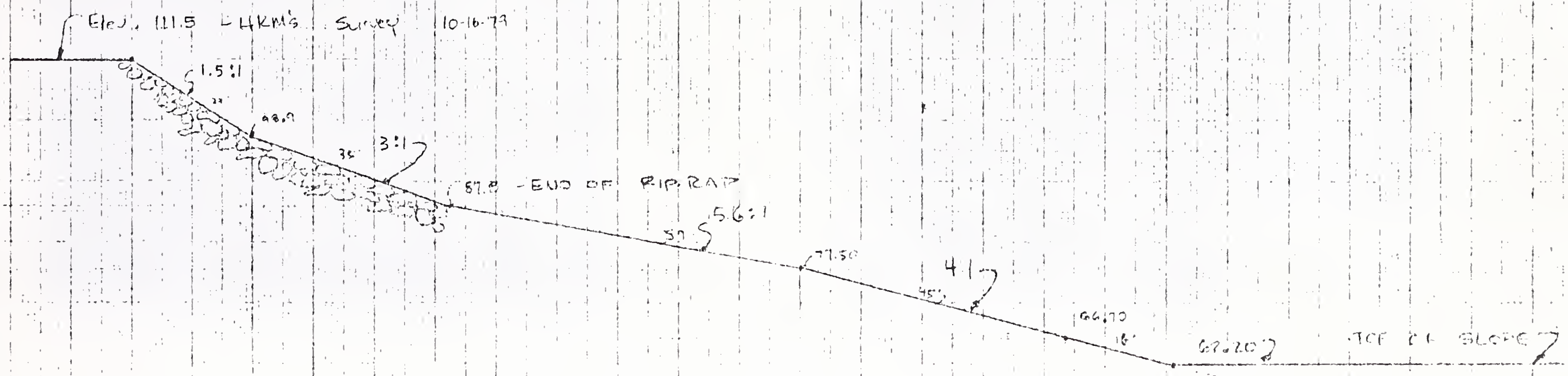
1. The gate stem in the lower gate tower should be repaired so that it can be fully opened.

2. The riprap on the upstream face should be re-shaped and properly bedded.

UPSTREAM FACE PETROLIA DAM - 11-12-80



UPSTREAM FACE - PETROLIA DAM 11-12-80
TYPICAL SECTION @ FENCE



Petrolia Dam
November 12, 1980

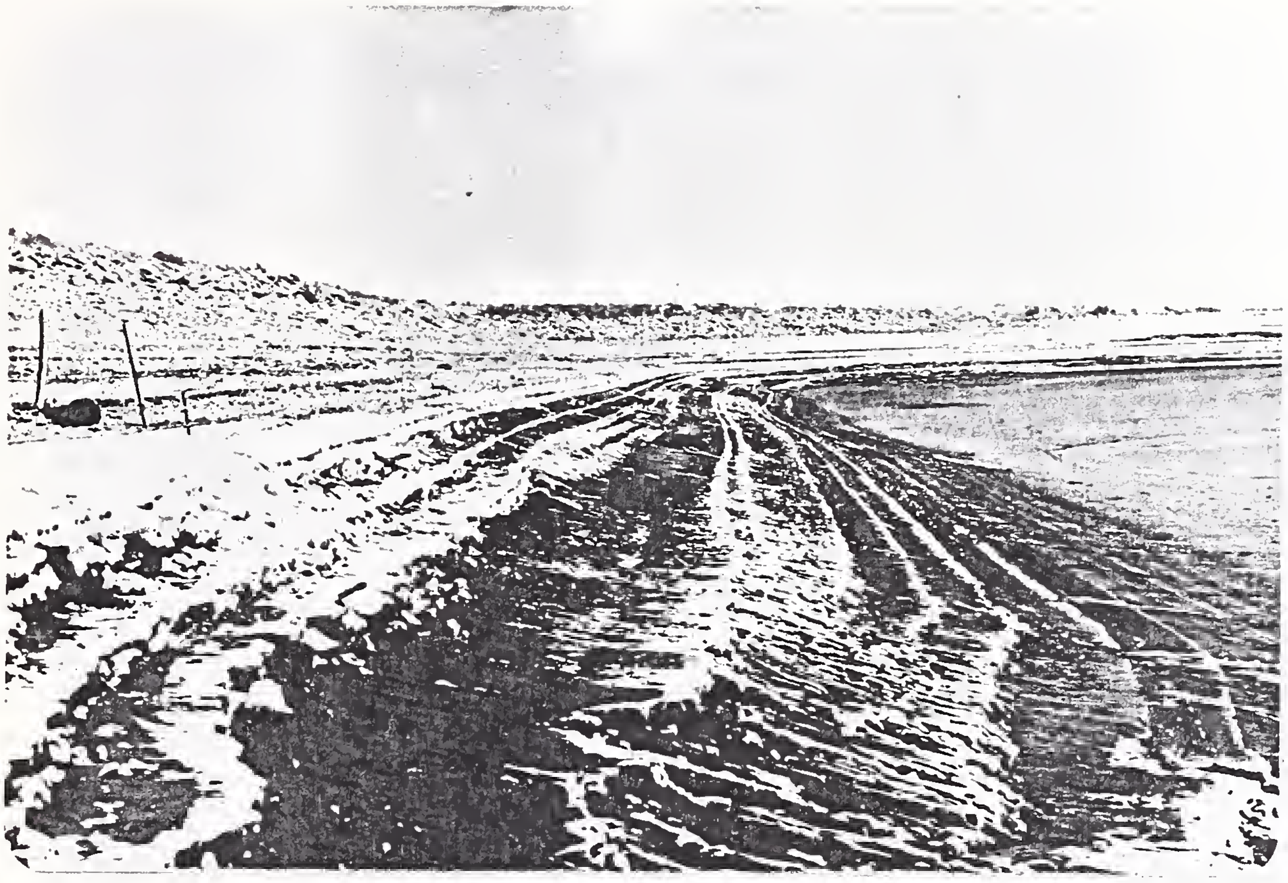


Photo 1. Upstream face of the dam from above the inlet. (Note beaching and scarfs).

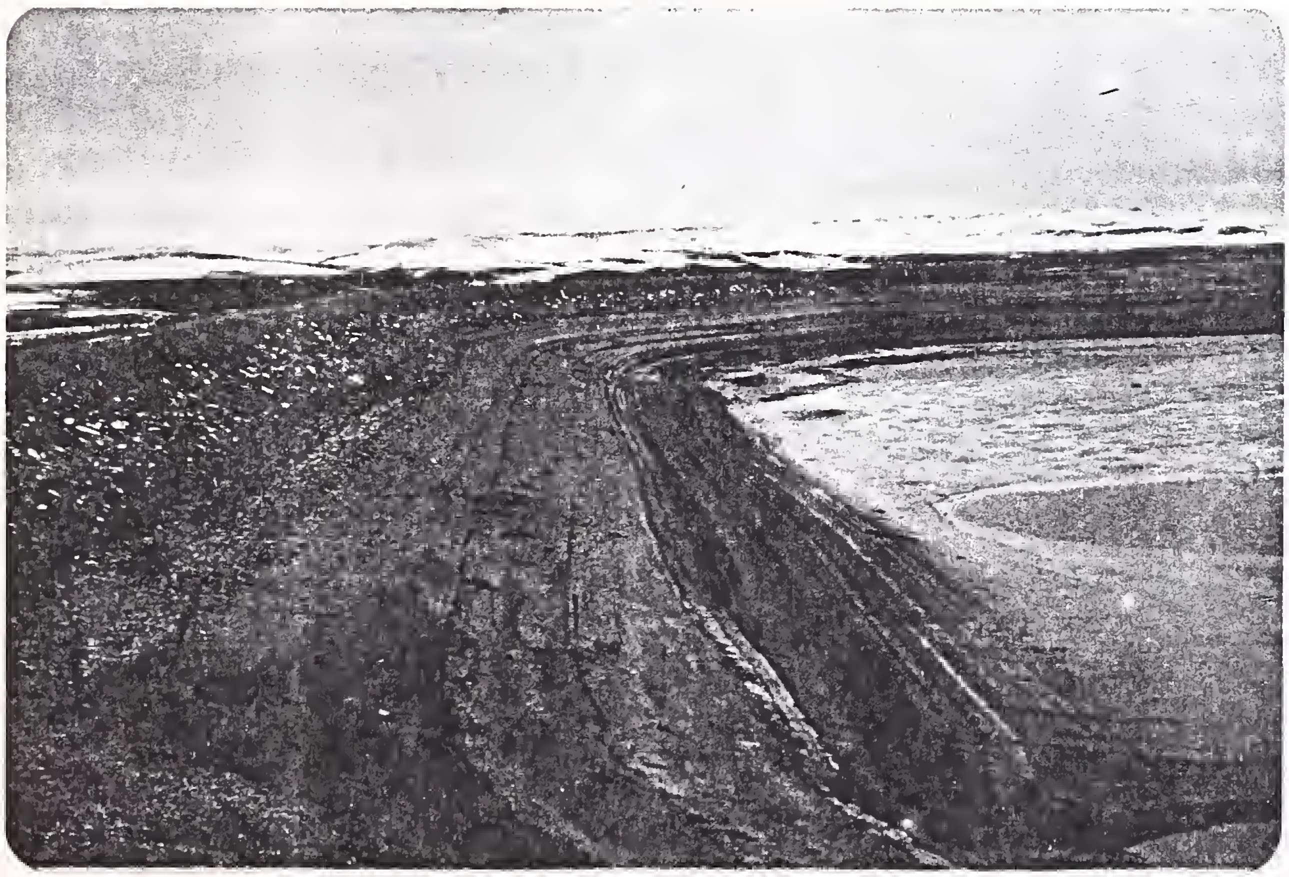


Photo 2. Upstream face from the left abutment. Inlet is in the lower right portion of the photo.

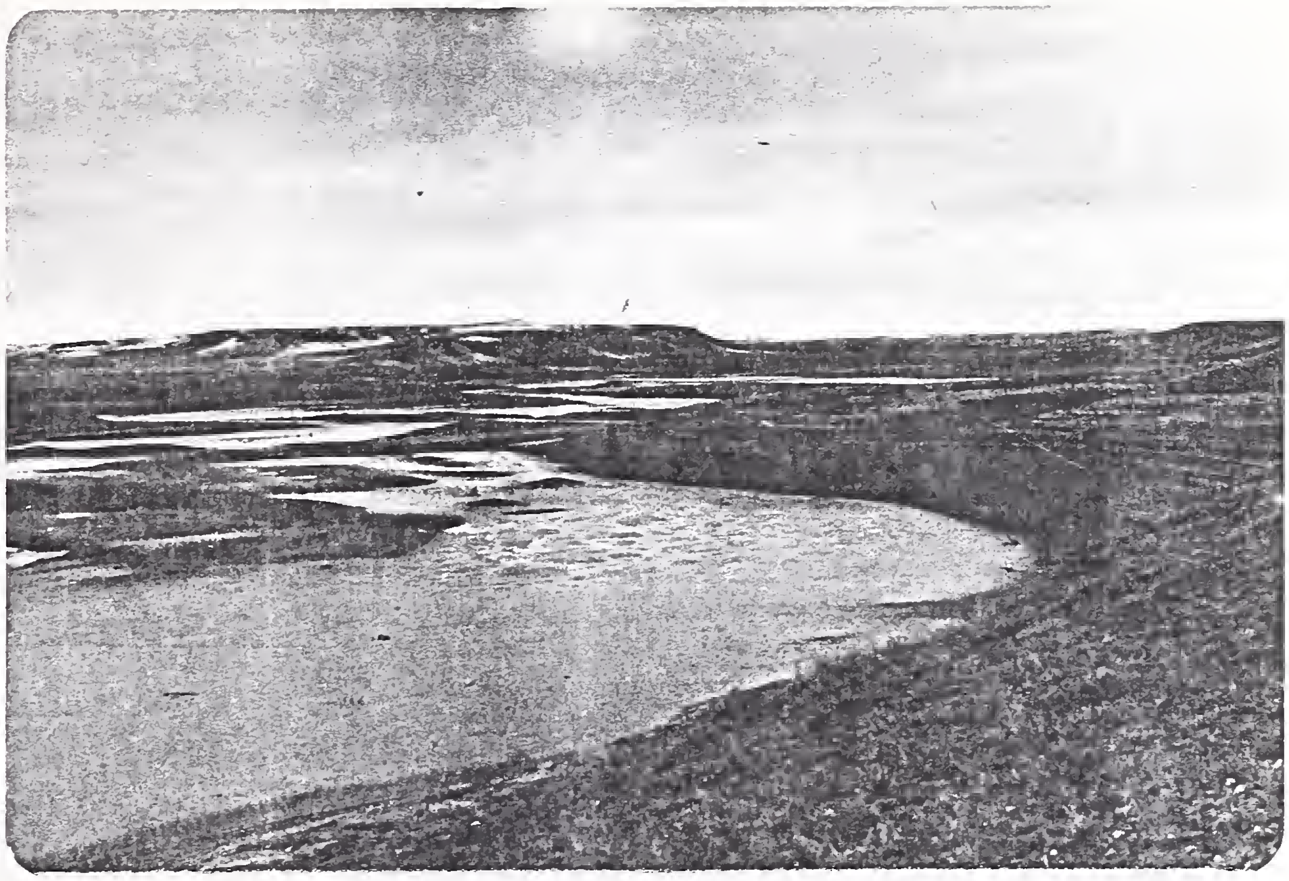


Photo 3. Part of the reservoir area on north abutment looking west.

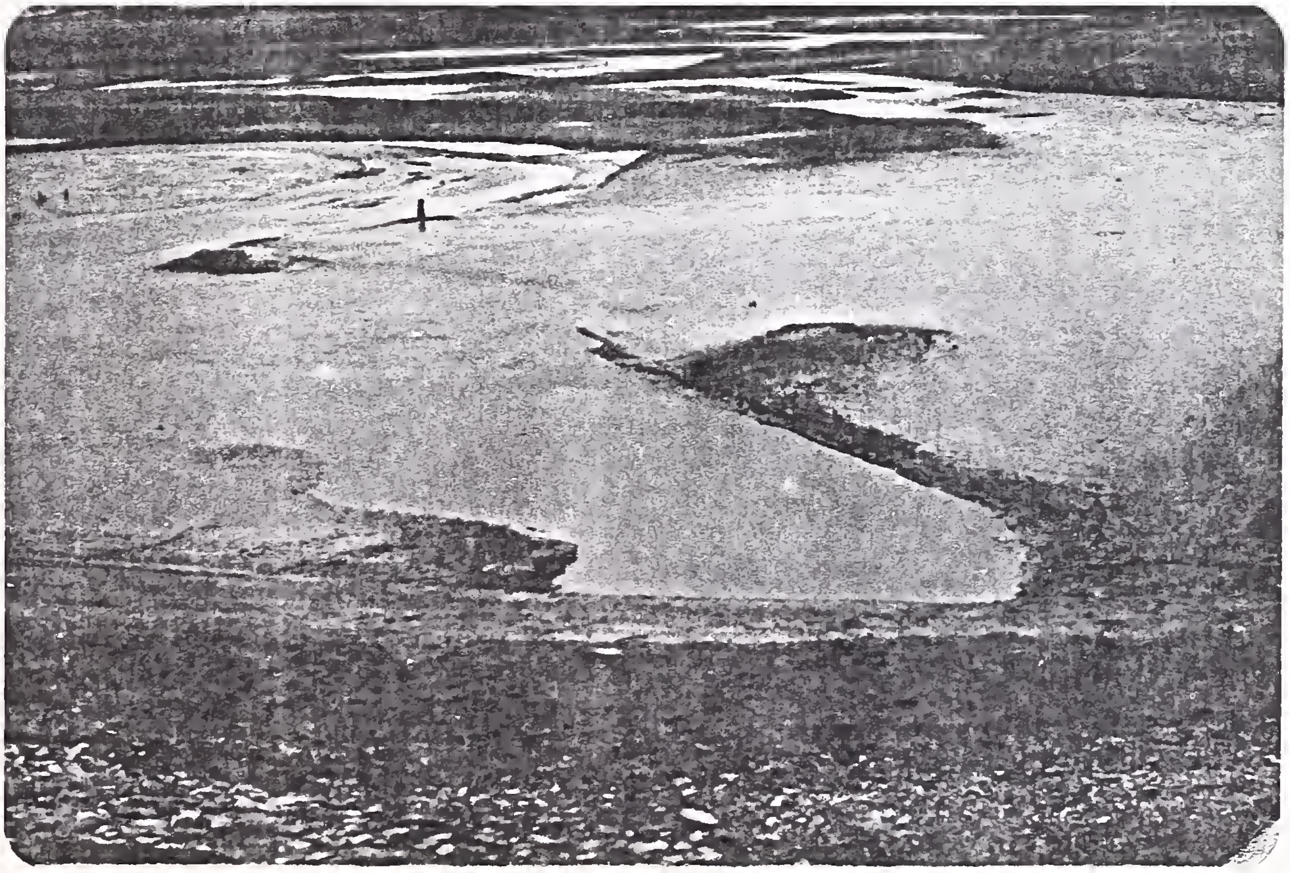


Photo 4. Part of the reservoir area near the inlet (near center)

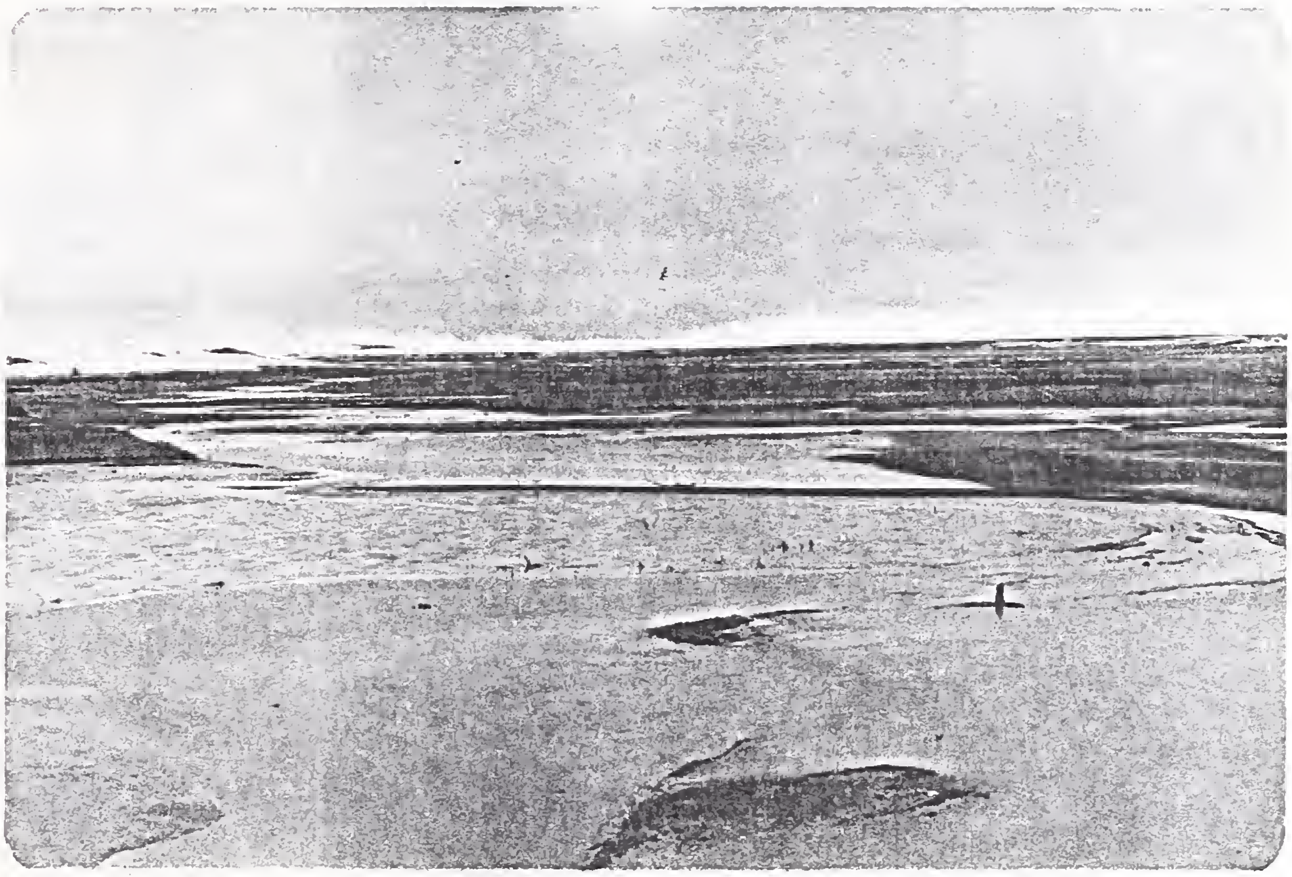


Photo 5. Part of the reservoir area looking towards the southwest. The Diversion Dike used for construction is near the center.

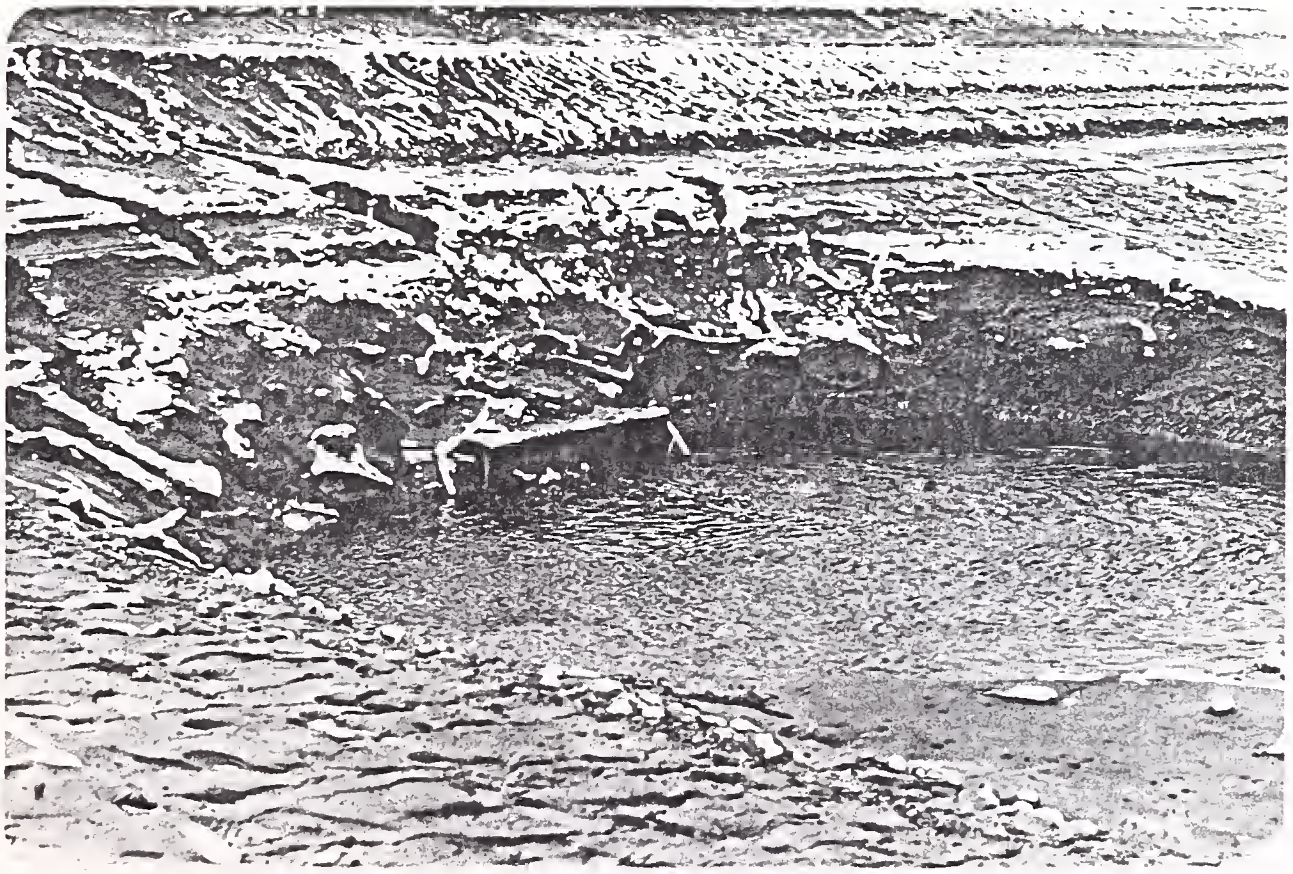


Photo 6. Looking at the inlet upon arrival. Note debris at water level on the inlet structure.

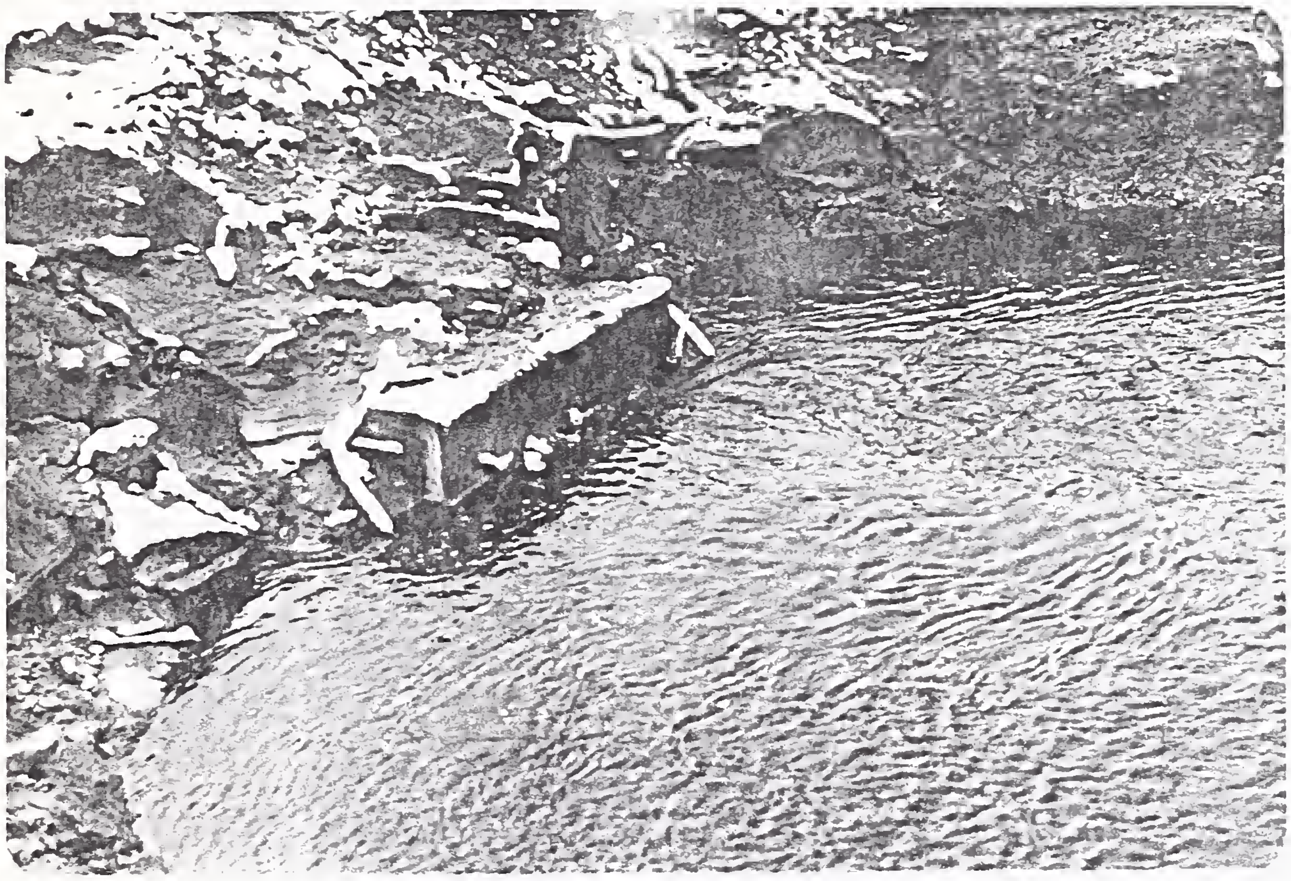


Photo 7. Inlet structure prior to cleaning debris.

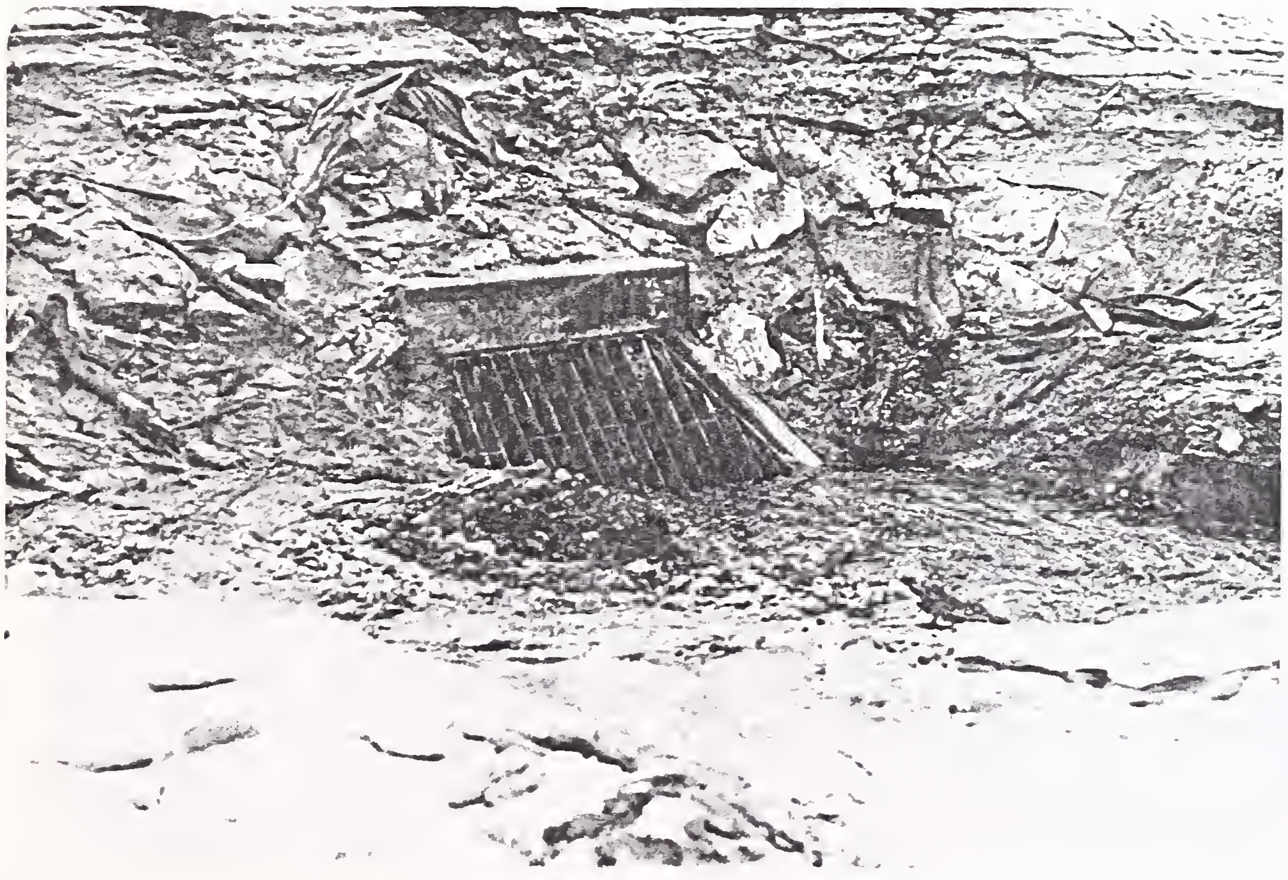


Photo 8. Inlet and trash rack after debris removed.

Photo 9. Inlet channel prior to cleaning the trash rack.

Photo 10. Inlet channel after the trash rack was cleaned.

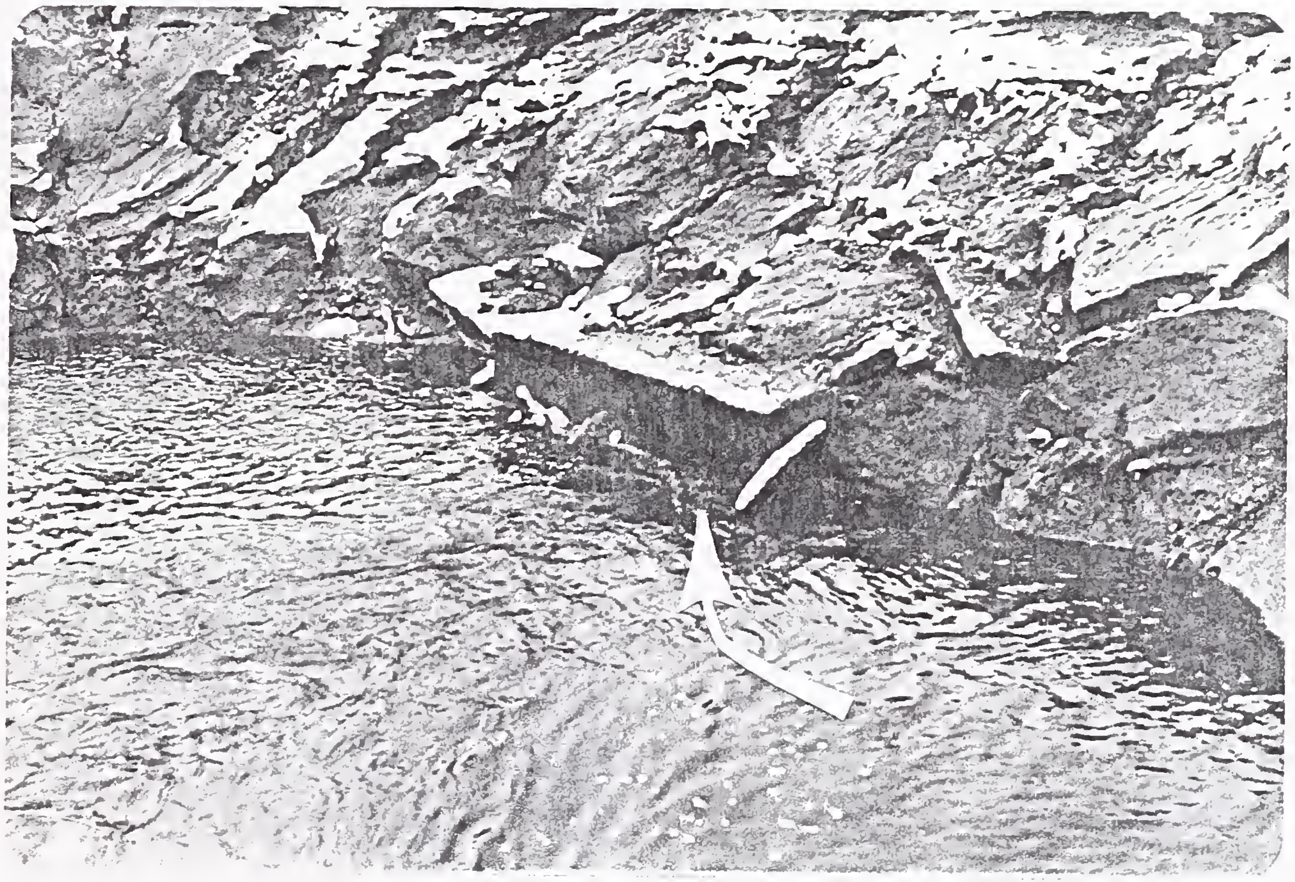


Photo 11. Additional photo of the inlet structure. Note debris against the head wall on top of the trash rack. Arrow points to where water was draining into the outlet.